Development of an optimised set of welded steel I-sections

by

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Declaration

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Synopsis

This thesis reports on the search for an optimal set of standard welded I-sections to replace the currently used set of hot-rolled and welded I-sections, listed in the Southern African Institute of Steel Constructions Red Book (SAISC, 2013).

Practical considerations rule out many sections. The set of I-sections should not be too large or too small. The problem is fairly unique amongst optimisation problems in that the search is not for a single section or group of sections that can meet the strength and stiffness requirements of a particular design problem at minimum cost. It is rather to find the set that will cover the whole design space, in particular the regions of the design space that are popular in the South African steel construction industry, most economically. The design space is defined by the spans, loads, conditions of lateral support, etc. that are encountered in real structures.

An optimisation methodology was developed for the purpose of obtaining an optimal set of welded I-sections to be used as simply supported unstiffened beams and girders. This optimal set was obtained from an initial set of sections (created according to available plate dimensions) by accounting for the practicality, capacity and popularity considerations of welded I-sections. Some key parameters were varied to obtain the most optimal set of welded I-sections, with each change in the parameter, providing a different optimal set. These different optimal sets were compared with a comparison methodology to obtain the best optimal set amongst all the optimal sets of welded I-sections that were developed.

This research project produced an optimal set of standardised welded I-sections for beams and girders, as intended. It is also demonstrated that it is economically viable to replace the currently used hot-rolled I-sections in South Africa with welded I-sections, and that the welded set can yield a weight saving of approximately 20 %. The research project also provides the basis for further research in the development of optimal sets of standardised sections, in order to include columns and the production cost of welded I-sections.

Samevatting

Hierdie tesis is ’n verslag van ’n soektog na ’n optimale stel standaard gesweiste I-profiele, wat bestaande warmgewalse en gesweiste I-profiele kan vervang, soos gelys in die Suider-Afrikaanse Instituut van Staalkonstruksie se Rooi Boek (SAISC, 2013).

Praktiese oorwegings skakel baie van die I-profiele uit. Die stel I-profiele moet nie te groot of te klein wees nie. Die probleem is redelik uniek onder optimeringsprobleme in die opsig dat die soektog is nie na ’n enkele I-profiel of ’n groep van I-profiele wat kan voldoen aan die sterkte en styfheid vereistes van ’n spesifieke ontwerp probleem teen die laagste koste nie, maar wel vir ’n stel wat die hele ontwerpruimte kan dek, en veral oor die populêre gebied van die ontwerpruimte, en dit so ekonomies moontlik doen. Die ontwerpruimte word gedefinieer deur die span, las, laterale ondersteuning, ens. wat voorkom in werklike strukture.

’n Optimiseringsmetode was ontwikkel met die doel om ’n optimale stel gesweiste I-profiele te identifiseer wat gebruik kan word as eenvoudig opgelegde balke. Hierdie optimale stel is verkry van af ’n aanvanklike stel I-profiele (geskep volgens beskikbare plaatafmetings) deur ’n proses wat verseker het dat die uiteindelike stel bestaan uit praktiese profiele met die vereiste kapasiteit en dat die populariteit van verskillende profiele in ag geneem word. Sommige van die parameters is gevarieer om die mees optimale stel gesweiste I-profiele te verkry. Vir elke stel parameters is ’n ander optimale stel verkry. Hierdie verskillende optimale stelle is dan met mekaar vergelyk deur ’n vergelykingsmetode toe te pas om die beste optimale stel van gesweiste I-profiele te verkry.

Hierdie tesis het ’n optimale stel standaard gesweiste I-profiele opgelewer vir gebruik as eenvoudig opgelegde balke, soos oorspronklik bedoel. Daar word ook getoon dat dit ekonomies lewensvatbaar is om die warmgewalsde I-profiele wat tans in Suid-Afrika gebruik word te vervang met gesweiste I-profiele, omdat die beste optimale stel ’n massaverlaging van ongeveer 20 % lewer. Hierdie tesis verskaf ook die basis vir verdere navorsing in die ontwikkeling van ’n optimale stel gestandaardiseerde profiele, met insluiting van kolomme en ’n ekonomiese evaluasie van die produksie van gesweiste profiele.

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Nomenclature

CE	Carbon Equivalent
SLS	Serviceability Limit State
ULS	Ultimate Limit State

A	Cross sectional area $[mm^2]$
A_p	Full cover surface $[mm^2]$
A_w	Web area $[mm^2]$
A_{ww}	Weld area $[mm^2]$
A_v	Shear area $[mm^2]$
a_w	Weld size $[mm]$
a_v	Weld thickness $[mm]$
B_r	Bearing resistance $[kN]$
b	Flange width $[mm]$
C_1	Welding technology parameter
C_2	Constant for a particular welding technology
C_e	Erection cost
C_F	Expected failure cost
C_f	Fabrication cost
C_I	Initial cost
C_m	Structural material cost
C_T	Total cost
C_t	Transportation cost
C_w	Warping constant $[mm^6]$
C_u	Ultimate compression force in member $[kN]$
C_y	Axial compression force in member $[kN]$
c_m	Steel cost per unit weight
E	Elastic modulus $[GPa]$
$f_{effective}$	Effective length weighting factor
f_{load}	Load weighting factor
f_s	Ultimate shear stress
$f_{spacing}$	Load spacing weighting factor
f_{span}	Span weighting factor
$f_{support}$	Lateral support weighting factor

f_u	Ultimate tensile strength $[MPa]$
f_y	Minimum yield stress $[MPa]$
f_{yf}	Minimum yield stress of flange $[MPa]$
f_{yw}	Minimum yield stress of web $[MPa]$
G	Shear modulus $[GPa]$
h	Section depth $[mm]$
h_w	Depth of the straight potion of web $[mm]$
h_w^*	Web depth $[mm]$
I	Moment of inertia $[mm^4]$
I_x	Moment of inertia about x-axis $[mm^4]$
I_y	Moment of inertia about y-axis $[mm^4]$
J	Torsion constant $[mm^4]$
K^*	Final ranking
K_f	Fabrication cost [$]
KL	Effective length $[m]$
K_M	Material cost [$]
K_{pi}	Cost of intumescent painting [$]
k_M	Material cost factor $[\$/kg]$
k_f	Fabrication cost factor $[\$/min]$
k_p	Cost of the specific painting [$]
k_{pi}	Additional cost of intumescent painting [$]
L	Section span $[m]$
L_c	Cutting length $[mm]$
L_w	Weld length $[m]$
l	Web length $[m]$
M	Total mass of a virtual project $[kg]$
$\bar{M}$	Unfactored total mass of a virtual project $[kg]$
M_a	Factored bending moment at one-quarter point of unbraced length $[kN.m]$
M_b	Factored bending moment at midpoint of unbraced length $[kN.m]$
M_c	Factored bending moment at three-quarter point of unbraced length $[kN.m]$
M_{cr}	Critical elastic moment $[kN.m]$
M_{max}	Maximum factored bending moment in unbraced length $[kN.m]$
M_{min}	Minimum factored bending moment in unbraced length $[kN.m]$
M_r	Moment resistance $[kN.m]$
M_{rx}	Moment resistance about the x-axis $[kN.m]$
m	Mass per unit length $[kg/m]$
N	Length of bearing $[mm]$
P	Factored point load $[kN]$
P_F	Probability of failure
P_{SLS}	Point load at serviceability limit state $[kN]$
R	Rating
R^*	factored rating

R_{min}	minimum rating
r	Radius of gyration $[mm]$
r_1	Radius of root fillet $[mm]$
r_2	Toe radius $[mm]$
r_x	Radius of gyration about x-axis $[mm]$
r_y	Radius of gyration about y-axis $[mm]$
$spacing$	load spacing $[m]$
T	Production time
T_{CG}	Times of hand cutting and machine grinding of strut ends
T_{CP}	Plate cutting and edge grinding time
T_{FP}	Time for flattening plates
T_{SP}	Surface preparation time
T_P	Painting time
T_{w1}	Preparation, assembly and tacking times
T_{w2}	Real welding time
T_{w3}	Additional fabrication action time
t	Plate thickness $[mm]$
t_w	Web thickness $[mm]$
t_f	Flange thickness $[mm]$
V	Volume of structure $[mm^3]$
V_{max}	Maximum applied shear force $[kN]$
V_r	Shear resistance $[kN]$
W	Weight of section $[kg/m]$
W_L	Weight of lightest section $[kg/m]$
w	Factored uniformly distributed load $[kN/m]$
w_{beam}	Own weight of beam $[kN/m]$
$w_{girderULS}$	Uniformly distributed load of girder at ultimate limit state $[kN/m]$
$w_{girderSLS}$	Uniformly distributed load of girder at serviceability limit state $[kN/m]$
w_{girder}	Own weight of the girder $[kN/m]$
w_{SLS}	Uniformly distributed load at serviceability limit state $[kN/m]$
w_{ULS}	Uniformly distributed load at ultimate limit state $[kN/m]$
x_u	Minimum weld ultimate tensile strength $[MPa]$
Z_{el}	Elastic section modulus $[mm^3]$
$Z_{el,x}$	Elastic section modulus about x-axis $[mm^3]$
$Z_{el,y}$	Elastic section modulus about y-axis $[mm^3]$
Z_{pl}	Plastic section modulus $[mm^3]$
$Z_{pl,x}$	Plastic section modulus about x-axis $[mm^3]$
$Z_{pl,y}$	Plastic section modulus about y-axis $[mm^3]$
Δ_{max}	Maximum displacement $[mm]$
ρ	Material density $[kg/mm^3]$
ρ_e	unit mass of steel $[7.85\ kg/m^3]$
θ	Weld angle

Θ_{df}	Difficulty factor for flattening plates
Θ_{ds}	Difficulty factor for surface preparation
Θ_{dp}	Difficulty factor for painting
Θ_{dw}	Difficulty factor for welding

Chapter 1

Introduction

1.1 Background

In South Africa, steel I-sections have historically been produced by hot-rolling. However, in recent years, South African steel fabricators have become increasingly interested in welded I-sections with similar profiles to those of hot-rolled I-sections. The reason for this is that EVRAZ Highveld Steel and Vanadium, the only producer of medium to heavy hot-rolled I-sections in South Africa (EVRAZ, 2013), ceased production on 20 July 2015 (Creamer Media Reporter, 2016) due to the shrinking of the South African steel industry and the increase in I-section imports, making some hot-rolled I-sections not readily available in South Africa.

Only ArcelorMittal still produces hot-rolled I-sections in South Africa, but their range is limited to IPE sections and non-standard UB sections up to a depth of 203 mm (ArcelorMittal South Africa, 2005). This means that hot-rolled I-sections, in the near future, may not be available in South Africa, other than from imports. This is likely to bring about unreliable supply and significant cost implications for I-sections in South Africa.

An alternative is the fabrication of welded I-sections, which could be cheaper than imported hot-rolled I-sections. In some countries, like China, a large volume of I-sections is already manufactured by welding plates together (Jiangyin Jianhe Steel Co. Ltd., 2016). According to the CEO of Union Steel, steel plates can be between R 1000 to R 2000 cheaper per ton than imported hot-rolled I-sections (refer to Appendix B). Steel plates are also readily available in South Africa, with ArcelorMittal South Africa Limited producing large volumes of steel plate at several of their factories.

Some companies like Steel Services (SAISC, 2016) and Pro Roof (Pro Roof, 2016) are already fabricating their own welded I-sections. According to SAISC (2016), Steel Services have already produced 30 tons of column (254 x 254 x 132) with their beam welding line. These were sections they could have bought from a steel mill, but the lead times of merchants overseas are long and the prices of I-sections are not fully confirmed until the material has arrived in South Africa, making welded I-sections a better option (SAISC, 2016).

Steel Services found that welded I-sections would cost 20 to 40 % more than the corresponding hot-rolled I-sections, depending on the weight of the I-section. The cost varies so much because the cost of welded I-sections depends both on the linear meters of welding and tons of steel, not only on the mass, as is normally the case with hot-rolled I-sections (SAISC, 2016).

The problem is, however, that Steel Services and other steel fabricators are fabricating welded I-sections with cross-sections of similar dimensions to the equivalent hot-rolled I-sections. Hot-rolled I-sections are limited to certain web to flange ratios, because of their manufacturing process, while welded I-sections are not. Welded I-sections can have thinner webs than hot-rolled I-sections, which makes them lighter and possibly more cost sufficient or just as cost effective as hot-rolled I-sections.

In principle, the process of fabricating welded I-sections allows engineers and steel fabricators great freedom to choose different proportions for each beam or column. It allows for a cross-section to achieve the most optimal strength and stiffness required for the specific application, with potential significant savings in material. In addition, stockholding can be reduced from having to keep large quantities of all types of I-sections to just keeping plates of various thicknesses.

However, in practice the designer and the system will find it difficult to deal with total freedom in respect of section geometry with no recourse to any standard set of welded I-sections. Communicating information on sections and weld sizes will be complex and fabrication and quality assurance will be difficult in a world where nothing is standardised. The option of calling on the steel manufacturer with the "name" of a certain profile will also not exist. Being able to communicate by simply using a code for a specific profile is of great value to industry. A standard set of welded I-sections is thus needed by engineers and steel fabricators should they opt to use welded I-sections in future.

With a standard set of welded I-sections, steel fabricators can also produce them in mass. Mass production will not only reduce the cost of welded I-sections, but also make welded I-sections readily available for engineers to use.

1.2 Problem Statement

The problem addressed in this research project is that the South African steel industry needs an optimal set of welded I-sections should they want to start production of welded I-sections in future; the term "optimal set" meaning a limited set that can serve the steel construction industry most economically. This would be a set of I-sections from which South African engineers can choose members for any of their structures, a set yielding an aggregate cost less than any other set of I-sections (welded or hot-rolled) with the same number of sections, for all the steel structures in the country.

Other researchers have optimised I-sections for specific applications in the past (refer to Section 2.7.1), but here we have a different problem. This problem is much more complex, as structures can take on almost any shape and structural elements can be required to do a great number of jobs. Adding to the difficulty is the fact that designers can make different assumptions while designing similar structures (refer to Chapter 7). Nevertheless, this research project strives to overcome all of these difficulties and aims to develop an optimal set of welded I-sections.

No evidence could be found that anyone has ever tried to solve this problem and there is thus a need for the research presented in this thesis.

Although the optimal set was developed with minimum weight as the optimality criterion, some researchers have shown that taking all factors that contribute to the cost into account yields more or less the same results (refer to Section 2.7.3.2). The optimisation method presented in this thesis can be extended to cover all aspects of cost (including welding, waste, etc.), but this was not done in this research project due to time constraints.

1.3 Objectives

The first and main objective of this research project is to produce an optimal set of standardised welded I-sections, ready for production by steel fabricators and to be documented in sources such as "The Red Book" (SAISC, 2013).

The optimal set of sections has to meet the general requirements of South African engineers, whilst being economical at the same time. It has to be optimal over the whole range of applications of such sections. The set must also be limited in number, mainly to contain the cost of stockholding and other costs associated with an unlimited number of choices.

The second objective of the project is to compare the optimal set of welded I-sections with the set of hot-rolled I-sections available in South Africa and to demonstrate how much cheaper welded I-sections could be in comparison with the hot-rolled I-sections historically used in South Africa.

1.4 Scope and limitations

This research project only focused on prismatic welded I-sections (including H-sections), with identical top and bottom flanges. The scope was limited to simply supported (pinned) beams and girders that support floors.

In this research project, secondary beams are referred to as BEAMS and primary beams are called GIRDERS. BEAMS are spaced between the ends of GIRDERS and support distributed loads, like floors, while GIRDERS span from column to column and support the BEAMS as point loads. Refer to Figure 1.1 for illustration.

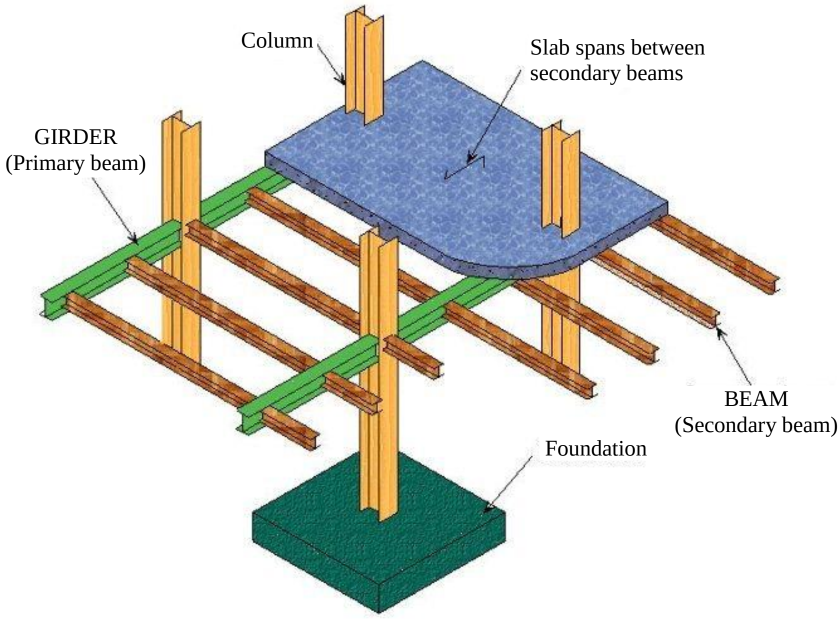


Figure 1.1: Definition of beams and girders (Khan, 2016)

The research project excluded:

- I-sections that are not used "every day", such as stiffened I-sections, castellated and cellular I-sections and I-sections with different flange sizes.
- Portal frames, as they can be fabricated more economically with tapered members, once one has decided to use welded sections.
- I-sections used for columns, because of time constraints.
- Composite construction, as only a small percentage of I-sections are used in composite construction in South Africa (refer to Chapter 7).
- Hybrid I-sections, as they are not cost effective for popular loads (refer to Section 2.7.3).

The main focus was the industrial application of I-sections, as that is where they are mostly used in South Africa. However, the application of I-sections in office buildings was also accounted for, as I-sections are also used in such buildings and could potentially be more economical to use than concrete. The use of I-sections in crushers, cranes, etc. was excluded from this research project.

The typical design parameters (spans, loads, lateral support, etc.) were obtained for South Africa. The optimal set of I-sections is therefore specifically developed for South Africa and could be different for other countries. The methodology presented in Chapter 5, can however be used in other countries for a similar exercise to this research project.

All the sections in the optimal set of I-sections had to meet all the requirements of SANS 10162-1 (SABS, 2011c) and SANS 10160-2 (SABS, 2011b), the applicable standard in South Africa when designing I-sections.

Only S355JR steel was considered in this research project, as it is the most commonly used structural steel in South Africa.

The I-sections in this research project were optimised according to weight and not cost. The costs of welding, waste, etc. were thus negated. Although the cost of welded I-sections is closely linked to their weight, full cost optimisation could give slightly different results.

1.5 Overview of thesis

This research project develops and implements an optimisation method to obtain an optimal set of standardised welded I-sections for South Africa.

Chapter 2 presents the literature study conducted for this project, with emphasis on optimisation and the use of I-sections in practice. It confirms that welded I-sections can outperform hot-rolled I-sections under particular conditions and that minimum weight optimisation of the welded I-sections will be sufficient to achieve the aim of this project. The literature study also found, as far as can be established, that nobody else has ever attempted to define an optimal set of welded I-sections.

Chapter 3 presents the requirements of practical welded I-sections in order to satisfy the demands of designers (structural engineers) and steel fabricators.

The requirements defined in Chapter 3 are applied in Chapter 4 to create large initial sets of practical I-sections, from which optimal sets can be select. One initial set was created by emulating all the hot-rolled I-sections available globally, while three other initial sets were generated based on the practical considerations obtained from Chapter 3.

The four initial sets were then optimised (Chapter 5) to obtain optimal sets of standardised welded I-sections, through a process involving seven steps. These steps ensured that the optimal set of welded I-sections contained economic solutions for the combinations of span, loading, lateral support, etc. that are frequently encountered in practice, while being limited to a reasonable number of sections. Chapter 5 also contains a flow diagram of the computer program that was developed and used to execute the optimisation methodology.

Chapter 6 provides all the equations that was used to calculate and confirm whether or not a welded I-section has sufficient capacity and displacement resistance at any point in the design space.

The field work conducted to determine which spans, loads, beam spacing, etc. are popular in practice is covered in Chapter 7. This information was used to calculate the popularity weighting factors used in the optimisation methodology in order to account for the fact that some design parameters will be more popular in practice than others.

Some parameters had to be varied in the optimisation process (Chapter 5) to be able to identify the most optimal set of welded I-sections and to test the sensitivity of the optimisation methodology. These parameters included different initial sets of welded I-sections, minimum ratings and sets of popularity weighting factors. This process produced a number of different optimal sets.

The different optimal sets are discussed in Chapter 8. It confirms that the optimisation methodology is not very sensitive to the popularity weighting factors, but quite sensitive to the size of the initial set of I-sections and the variation in minimum rating.

The different optimal sets were compared to each other with the use of a comparison methodology (Chapter 9) to determine which optimal set performs the best. The methodology compares the various sets to each other on the basis of the total mass of steel required to build a virtual project. The virtual project aims to correspond to the average of all the structures built in South Africa over a one year period.

The thesis is closed off with a summary of the conclusions (Chapter 10) gathered throughout this research project, including recommendations for future research in this regard.

The work in this thesis makes a unique contribution to the body of knowledge in the field of steel construction. In as far as can be established this is the first effort to define an optimised set of standardised welded I-sections. No evidence could be found of any earlier research into the actual needs of the steel construction industry with respect to the practical problems that need to be addressed or the frequency of occurrence of each problem in South Africa.

Chapter 2

Literature study

2.1 Introduction

The topic of this thesis, "Development of an optimised set of welded steel I-sections", deals with two main issues, namely optimisation and the use of I-sections in practice.

Optimisation is a general term and is defined differently from one researcher to the next, but ultimately they all mean more or less the same. For example, Goble and Fred (1971) referred to optimisation as the design methods which select the best design for the specified limitations using automated search techniques. Farkas and Jármai (2013), on the other hand, defined optimisation as: "a search for better solutions, which fulfil the requirements better" (Farkas and Jármai, 2013).

The optimisation definition of Farkas and Jármai (2013) was used in this research project, as none of the standard optimisation algorithms was used in this thesis. Section 2.6 provides the reasons why such a algorithm was not used.

An I-section can either be a hot-rolled I-section or a welded I-section. This thesis, however, only focuses on welded I-sections and how they compares with hot-rolled I-sections.

2.2 I-sections in general

The following subsections provide insight into how I-sections are used and how they are fabricated.

2.2.1 Hot-rolled I-sections

Hot-rolled I-sections are manufactured at steel mills through a hot-rolling process. The hot-rolling process moulds billets into hot-rolled I-sections with rollers, as shown in Figure 2.1.

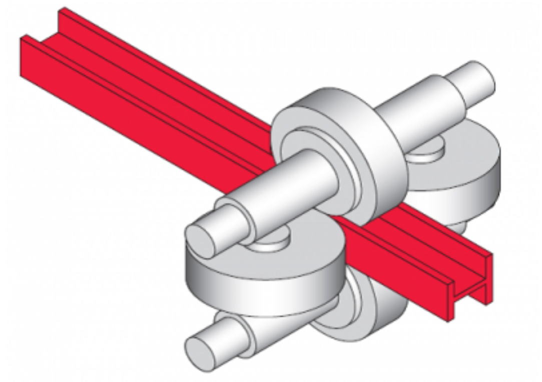


Figure 2.1: Rolling I-sections in steel mill (Steelconstruction.info, 2016a)

As mentioned in Section 1.1, there is only one company in South Africa that still produces hot-rolled I-sections, namely ArcelorMittal. They only produce IPE sections and non-standard UB sections up to a depth of 203 mm (ArcelorMittal South Africa, 2005).

EVRAZ Highveld Steel and Vanadium also produced hot-rolled I-sections in South Africa, but ceased production on 20 July 2015 (Creamer Media Reporter, 2016). They were the only producer of medium to heavy hot-rolled I-sections in South Africa and without them South Africa was forced to import I-sections or find an alternative, as mentioned in Section 1.1 (EVRAZ, 2013).

South Africa has a set of 63 hot-rolled I-sections, specified in The Red Book (SAISC, 2013) and used by engineers. This set consists of IPE sections, universal beams, taper flange I-sections and universal columns. All of these sections were manufactured in South Africa, but at the moment only IPE and non-standard UB sections are still produced.

Globally, there are numerous sets of standard hot-rolled I-sections. These are discussed in Section 4.7.1.

2.2.2 Welded I-sections

Welded I-sections are often referred to as plate girders and are fabricated by welding three plates together, two flange plates and a web plate.

Welded I-sections can be fabricated by any qualified steel fabricator. Steel Services (SAISC, 2016) and Pro Roof (Pro Roof, 2016) are two South African companies that have the necessary specialised equipment, an automated beam welding line, to produce welded I-sections. They are, however, not producing welded I-sections in mass, as some companies do in China, but only specific beams as needed.

The welded beam lines differ from company to company. Figure 2.2 shows two different machines that can be used to fabricate welded I-sections. The machines shown can be obtained from Shining Industrial Enterprise (China) Co. Ltd. (2016) and Sinotech Machinery Co. Limited (2016).

(a) Gantry welding line

(b) H-beam welding line

Figure 2.2: Different automated beam welding lines (Shining Industrial Enterprise (China) Co. Ltd., 2016; Sinotech Machinery Co. Limited, 2016)

The gantry welding line (Figure 2.2a) is usually used to fabricate larger plate girders, while the H-beam welding line (Figure 2.2b) is usually used to fabricate welded I-sections, with more or less the same properties as hot-rolled I-sections. Most of the companies in South Africa make use of gantry welding machines or a machine similar to it.

2.2.3 Comparison of hot-rolled and welded I-sections

2.2.3.1 Cross section

The cross section of a hot-rolled I-section is limited by the manufacturing process. Its web depth is limited by the available rollers in the steel mill (refer to Figure 2.1) and its web to flange thickness ratios are limited by practical manufacturing constraints. Not one of the hot-rolled I-sections available globally, from ArcelorMittal's website (ArcelorMittal, 2016), has a web to flange thickness ratio smaller than 0.5.

The cross section of a welded I-section does not have web depth or web to flange thickness ratio limits and will normally be deeper than the comparable hot-rolled I-section (Hoadley, 1964), but it does have a few other fabrication limitations. The flange and web thicknesses of welded I-sections are limited by the available plate sizes (refer to Section 4.3.1), the minimum weldable plate thickness of 5 mm (refer to Section 3.5.2) and the maximum web slenderness ratio of $83000/f_y$ (SABS, 2011c).

Hot-rolled IPE sections can have web and flange thicknesses of as little as 3.6 mm and 4.5 mm, respectively (SAISC, 2013). It is unlikely that lighter welded profiles can be made. Medium to heavy welded I-sections, however, can always be fabricated to be lighter than hot-rolled I-sections, because they can have smaller web to flange thickness ratios.

2.2.3.2 Section cost

The savings in material due to the better proportions of welded I-sections can be outweighed by increased fabrication costs (Bresler et al., 1960; Hoadley, 1964).

In the 1960's, hot-rolled I-sections of any grade of steel were more economical than corresponding welded I-sections in most cases (Bresler et al., 1960). However, since then the fabrication cost of welded I-sections has reduced over the years with better fabrication technology. Some companies, like Jiangyin Jianhe Steel Co. Ltd. from China (Jiangyin Jianhe Steel Co. Ltd., 2016), are already producing welded I-sections in mass.

As mentioned in Section 1.1, Steel Services (a South African company) found that welded I-sections would cost between 20 % and 40 % more per ton than the corresponding hot-rolled I-sections, depending on the weight of the I-section. The cost difference found by Steel Services varies so widely because the cost of welded I-sections depends on both the linear meters of welding and the mass; and not only on the mass, as is normally the case with hot-rolled I-sections (SAISC, 2016). The welded I-sections could be much cheaper if Steel Services used optimal (lighter) cross sections. If welded I-sections are produced in mass the production cost can also be reduced.

Welded I-sections will never be cheaper than hot-rolled I-sections for really small beams (IPE's), as small hot-rolled I-sections can be manufactured lighter (refer to Section 2.2.3.1) and at less cost. Welded I-sections can, however, be cheaper for medium to heavy I-sections if the savings in material outweighs the increase in fabrication cost. According to Bresler et al. (1960), in the 1960's hot-rolled I-sections were more economical for spans below 30 ft (9.1 m) and welded I-sections above 70 ft (21.3 m), for ordinary loads. Even if medium to heavy welded I-sections cost more to fabricate, the import costs and time delays should also be considered for South Africa, which could make them preferable in South Africa.

For heavier construction, welded I-sections are the obvious choice, as the available hot-rolled I-sections do not have sufficient strength (Bresler et al., 1960; Hoadley, 1964).

2.3 Optimisation in general

The optimisation of welded I-sections falls under the topic general structural optimisation. According to Save and Prager (1985), structural optimisation consists traditionally of developing an adequate number of alternative designs and selecting the best of the developed designs. This method of optimisation is normally laborious, but the designer could look for characteristic features that would enable him to obtain an economical design without exploring unnecessary alternatives. Alternatively, automated search techniques can be used to obtain the best solution (Save and Prager, 1985).

All the developed structural designs should always meet some standards and practical requirements, which are normally specified by the structural engineer, but can also be specified by the client or architect. In modern structural engineering, the main requirements of a structure are as follows (Farkas and Jármai, 2013):

- load-carrying capacity (safety)
- serviceability (displacements and vibrations)
- manufacturability (constructability)
- economy

According to Farkas and Jármai (2013), all optimisation systems can be symbolised by the spatial structure shown in Figure 2.3. When one of the requirements is not met, the system will not work well (Farkas and Jármai, 2013).

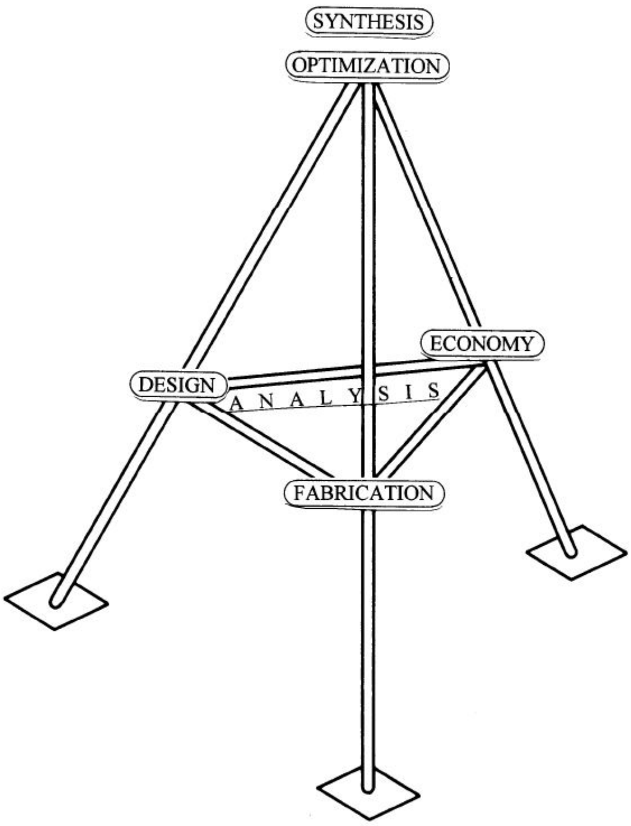


Figure 2.3: Spatial structure of a structural optimisation system (Farkas and Jármai, 2013)

These requirements can be fulfilled with the use of a structural optimisation system, in which the cost of the structure is minimised with the use of a cost function (i.e. minimum cost or minimum weight) while considering the design (load-carrying capacity and serviceability requirements) and manufacturability constraints (Farkas and Jármai, 2013).

Mela and Heinisuo (2014) found the solution produced by minimum cost optimisation, using a cost function, is not much different from the solution obtained with minimum weight optimisation (refer to Section 2.7.3.2). If this is true, minimum cost optimisation is not worth the amount of work required to produce an accurate solution. In addition, cost optimisation can easily produce wrong results with the wrong information and a great deal of research is required to produce accurate cost functions.

2.4 Minimum weight optimisation

Kanagasundaram and Karihaloo (1983) defined minimum weight optimisation very clearly. According to them, the term "optimal", in minimum weight optimisation, is the design which requires the smallest quantity of material to attain the required strength and stiffness under the defined loading conditions.

Minimum weight optimisation is the optimisation method used most in engineering practice, as it is less time consuming and requires less information than minimum cost optimisation. In addition, the costing of a structure is normally based on tons of material. A minimum cost optimisation system is, however, normally held in higher regard than minimum weight optimisation (Farkas and Jármai, 2013), as the cost of a structure does not just include the cost of material, but also labour, welding consumables, etc.

2.5 Minimum cost optimisation

2.5.1 Introduction

With the advancement of technology, the popularity of minimum cost optimisation has increased over the years. Some even go so far as to say that it is not good enough for a structure just to be optimised with respect to weight any more (Farkas and Jármai, 2013). While weight is a major component of the total cost of a steel structure, the minimisation of cost should be the final objective.

Sarma and Adeli (1999) summarised the research done on cost optimisation of steel structures published in research journals. They found that cost optimisation can lead to additional savings in cost of between 7 % and 26 %, when compared to minimum weight optimisation.

There are numerous research studies available on cost optimisation and cost functions, for example Farkas and Jármai (2013). Most of the cost functions were developed to be used on a number of different structures, but some researchers, like Mela and Heinisuo (2014), have already developed cost functions specifically for welded I-sections (refer to Section 2.7.2.2).

Farkas and Jármai (2013) developed one of the most recent cost functions. It included the material cost, as well as the fabrication cost. The cost function can also include assembly cost if needed, but the cost of transportation and erection can be omitted, according to Farkas and Jármai (2013), since their influence is small.

According to Farkas and Jármai (2013), it is also very important to include fabrication constraints as part of the design constraints. The aim is always to bridge the gap between fabrication practice and the optimisation theory when research is needed for practice, which is also the case in this thesis (Farkas and Jármai, 2013).

The book written by Farkas and Jármai (2013) is based on their previous paper (Jármai and Farkas, 1999). Jármai and Farkas (1999) developed one of the first complete cost functions for welded steel structures (refer to Section 2.5.4). The cost function accounts for the welding times of various welding technologies and for other fabrication times like flattening plates, cutting, surface preparation, electrode changing, deslagging, painting, etc. (Jármai and Farkas, 1999). Their 2013 edition (Farkas and Jármai, 2013) only accounted for a few additional costs.

The cost functions can also be grouped into different methods of cost optimisation. Deterministic and reliability-based cost optimisation are two of these methods. According to Sarma and Adeli (1999), the majority of cost optimisation papers published deal with deterministic cost optimisation.

2.5.2 Deterministic cost optimisation

Deterministic cost optimisation refers to structural optimisation performed over a predefined set of loading conditions according to code-specified requirements. The general deterministic cost function for steel structures can be defined as follows (Sarma and Adeli, 1999):

$$C_T = C_m + C_f + C_t + C_e \tag{2.1}$$

where C_T is the total cost, C_m is the structural material cost (structural members), C_f is the fabrication cost, C_t is the transportation cost and C_e is the erection cost.

The structural material cost (C_m) includes beams, columns and bracing; or only one of these elements, depending on the problem. The fabrication cost (C_f) includes the material cost of connection elements (including bolts and electrodes) and labour cost. The transportation cost (C_t) is the cost of transporting the structural members to site (Sarma and Adeli, 1999).

The structural material cost can be calculated with the following equation, according to Sarma and Adeli (1999):

$$C_m = c_m \cdot \rho_s \cdot V = c_m \cdot W \tag{2.2}$$

where c_m is the steel cost per unit weight, ρ_s is the steel unit weight, V is the volume of the structure and W is the total structural weight.

Sarma and Adeli (1999) also presented information specifically applicable to the cost optimisation of beams and plate girders (refer to Section 2.7.2.2).

2.5.3 Reliability-based cost optimization

In reliability-based cost optimisation, the resistances and loads are considered randomly and the optimisation process is performed for a given probability of exceeding the structural capacity, which can also be given in terms of a safety factor (Sarma and Adeli, 1999).

The reliability factor can be considered directly or indirectly in the cost optimisation process (Sarma and Adeli, 1999).

2.5.3.1 Direct approach

When the direct approach is used, the reliability factor is included directly into the cost function, as shown in the following cost function (Sarma and Adeli, 1999):

$$C_T = C_I + P_F \cdot C_F \tag{2.3}$$

where C_T is the total cost of the structure, C_I is the initial cost, P_F is the probability of failure and C_F is the expected failure cost.

The probability of failure (P_F) represents the risk of the loading exceeding the structural capacity. The expected failure cost (C_F) includes the structure replacement cost and the cost of casualties, business interruption, etc. The two mentioned parameters are extremely difficult to determine, especially if human lives are endangered (Sarma and Adeli, 1999).

2.5.3.2 Indirect approach

In the indirect method the reliability term is considered separately from the cost function. The cost function is thus only the initial cost (C_I), refer to Equation 2.3. The reliability term is considered in the form of a constraint, such as (Sarma and Adeli, 1999):

$$P_F \leq (P_F)_{allowable} \tag{2.4}$$

where $(P_F)_{allowable}$ is the probability of failure.

The deterministic optimisation process (mentioned in Section 2.5.2) can thus be converted to a indirect reliability-based optimisation process by adding probability constraints.

2.5.4 Cost function developed by Farkas and Jármai (2013)

The following information about the cost function developed by Farkas and Jármai (2013) is presented in this thesis to show how complex the cost function and minimum cost optimisation of steel structures can be, as well as how easily an error can occur when the wrong information is used in developing the cost function.

The cost function developed by Farkas and Jármai (2013) was developed for deterministic cost optimisation (refer to Section 2.5.2) and was based on the work of Jármai and Farkas (1999). Some of the information presented in this section was obtained from Jármai and Farkas (1999) in order to explain the reasoning behind the cost function developed by Farkas and Jármai (2013).

The cost function developed by Farkas and Jármai (2013) can be used for numerous metal structures and includes the cost of material and fabrication. The fabrication costs include cost of cutting, edge grinding, shell forming, assembly, welding, surface preparation and painting; and were formulated according to the fabrication sequence. Other costs like transportation and maintenance were not considered in the cost function (Farkas and Jármai, 2013).

2.5.4.1 Fabrication cost factor

It is very difficult to obtain fabrication cost factors which are globally valid, due to the difference in technology available in different countries and industries. There are also large differences in the fabrication cost factors for different companies in one country, for the same reasons (Jármai and Farkas, 1999).

Fabrication cost is directly linked to fabrication time. Fabrication time is dependent on the available technology and the level of manufacturing. After obtaining or computing the fabrication time of each fabrication phase, the fabrication cost can be determined by multiplying by the specific cost factor (Jármai and Farkas, 1999). Jármai and Farkas (1999) defined the fabrication cost function as follows:

$$K = K_M + K_f = k_M \cdot \rho \cdot V + k_f \cdot \sum_i T_i \tag{2.5}$$

where K_M is the material cost, K_f is the fabrication cost, k_M is the material cost factor, k_f is the fabrication cost factor, ρ is the material density, V is the volume of the structure and T_i is the production time for each aspect i.

2.5.4.2 Total cost function

The total cost function of Farkas and Jármai (2013) is more or less the same as Jármai and Farkas (1999). The only difference is that Farkas and Jármai (2013) consider more types of fabrication technology. The total cost function developed by Farkas and Jármai (2013) is as follows and was derived from Equation 2.5:

$$\frac{K}{k_M} = \rho \cdot V + \frac{k_f}{k_M} \cdot (T_{w1} + T_{w2} + T_{w3} + T_{CP} + T_{FP} + T_{SP} + T_P + T_{CG} + ...) + \frac{K_{pi}}{k_m} + ... \tag{2.6}$$

where all the T components correspond to the T_i shown in Equation 2.5 and K_{pi} the cost of intumescent painting. The definition of the other symbols will follow in the following subsections. Sections 2.5.4.3, 2.5.4.4 and 2.5.4.11 defines the cost factors and Section 2.5.4.5 to 2.5.4.10 the T components.

Fabrication times may be similar, but fabrication costs will differ from country to country. Jármai and Farkas (1999) introduced the fabrication and material specific cost ratio (k_f/k_M) which varies between 0 and 2 kg/min. The cost ratio makes it possible to build the cost function from the different fabrication times and to optimise steel structures in different economic conditions (Jármai and Farkas, 1999). If $k_f/k_M = 0$, we obtain the mass minimum. $k_f/k_M = 0.5$ can be used for developing countries, k_f/k_M = 1.0 - 1.5 for West European labour and k_f/k_M = 2.0 for very high labour costs, as in Japan and the USA. Even if the production costs are similar for countries, the difference in labour costs can be significant (Farkas and Jármai, 2013).

2.5.4.3 The cost of material

The cost of material can be calculated using the following equation (Farkas and Jármai, 2013):

$$K_M = k_M \cdot \rho \cdot V \tag{2.7}$$

where K_M [\$] is the material cost, k_M [$\$/kg$] is the corresponding material cost factor, ρ [kg/mm^3] is the density of the material and V [mm^3] is the volume of the structure.

Farkas and Jármai (2013) specify material cost factors and densities for a few different materials. However, only the factors corresponding to steel are presented in this section, as only welded steel I-sections were considered in this research project. The specific material cost for steel can be k_M = 1.0 - 1.3 $\$/kg$ and density $\rho = 7.85 \times 10^{-6} kg/mm^3$ (Farkas and Jármai, 2013).

2.5.4.4 The fabrication cost in general

The general fabrication cost can be calculated using the following equation (Farkas and Jármai, 2013):

$$K_f = k_f \cdot \sum_i T_i \tag{2.8}$$

where K_f [\$] is the fabrication cost, k_f [$\$/min$] is the corresponding fabrication cost factor (k_f = 0 - 1 $\$/min$) and T_i [min] are the production times (Farkas and Jármai, 2013).

According to Farkas and Jármai (2013), it can be assumed that the k_f value is constant for a given manufacturer. Accordingly k_f could be ignored for this thesis, as the cost function will be used to compare I-sections from the same steel fabricator.

2.5.4.5 Fabrication times for welding

The main times associated with welding are the preparation, assembly, tacking, time of welding, changing electrodes, deslagging and chipping (Farkas and Jármai, 2013).

Preparation, assembly and tacking times (T_{w1})

The sum of preparation, assembly and tacking times (T_{w1}) can be calculated as follows (Farkas and Jármai, 2013):

$$T_{w1} = C_1 \cdot \Theta_{dw} \cdot \sqrt{\kappa \cdot \rho \cdot V} \tag{2.9}$$

where C_1 is the welding technology parameter (usually equal to 1), Θ_{dw} is the difficulty factor and κ is the number of structural elements which will be assembled.

The difficulty factor (Θ_{dw}) is used to express the complexity of the structure and depend on the kind of structure (Farkas and Jármai, 2013). The difficulty factors range between 1 and 4, and the difficulty factors for long welds in a flat positions are as follows (Jármai and Farkas, 1999):

- $\Theta_{dw} = 1.0$ for V-welds at 60^o
- $\Theta_{dw} = 2.0$ for fillet welds at 90^o

These difficulty factors correspond to the welds used in the fabrication process to fabricate welded I-sections.

Real welding time (T_{w2})

The real welding time can be calculated with the following equation (Farkas and Jármai, 2013):

$$T_{w2} = \sum_i C_{2i} \cdot a_{wi}^2 \cdot L_{wi} \tag{2.10}$$

where C_{2i} is the constant for a particular welding technology, a_{wi} is the weld size and L_{wi} is the weld length. C_2 accounts for both the welding technology and the time difference between positional (vertical and overhead) and normal downhand position welding. According to Jármai and Farkas (1999), $C_2 = 0.8 \times 10^{-3}\ min/mm^{2.5}$ for manual-arc welding and $0.5 \times 10^{-3}\ min/mm^{2.5}$ for CO_2-welding.

The equations for different welding technology can be found in Table 2.1. The explanation of the different welding terms can be obtained from Table 2.2.

Table 2.1: Welding times T_{w2} (min/mm) in the function of weld size a_w (mm) for longitudinal fillet welds, downhand position (Farkas and Jármai, 2013)

Welding technology	a_w **[mm]**	$10^3 T_{w2} = 10^3 C_2 a_w^2$
SMAW	0-15	$0.7889 a_w^2$
GMAW-CO2	0-15	$0.3394 a_w^2$
SAW	0-15	$0.2349 a_w^2$

Table 2.2: Welding technology term explanation (Farkas and Jármai, 2013)

Abbreviations	**Descriptive terms**
SMAW	Shielded Metal Arc Welding
GMAW-CO2	Gas Metal Arc Welding with CO2
SAW	Submerged Arc Welding

Additional fabrication action time (T_{w3})

Additional fabrication actions include changing the electrode, deslagging and chipping. Farkas and Jármai (2013) approximated the additional fabrication time as follows:

$$T_{w3} = 0.3 \cdot \sum_i C_{2i} \cdot a_{wi}^n \cdot L_{wi} \tag{2.11}$$

It is proportional to T_{w2}. The sum of the two time elements (T_{w2} and T_{w3}) is as follows:

$$T_{w2} + T_{w3} = 1.3 \cdot \sum_i C_{2i} \cdot a_{wi}^n \cdot L_{wi} \tag{2.12}$$

where T_{w2} is the real welding time, T_{w3} the additional welding time, C_{2i} is the constant for a particular welding technology, a_{wi} is the weld size, L_{wi} is the weld length and n is the value obtained by curve fitting calculations (refer to Table 2.1).

2.5.4.6 Cutting and edge grinding times

Oxy-fuel gas, plasma, laser and abrasive waterjet cutting are the four most commonly used non-contact methods of steel cutting. The abrasive waterjet cutting method cuts by abrasive erosion. The other three cutting methods are thermal in nature. These methods are mostly used to make precision interior and external cuts on plate and flat sheet material (Farkas and Jármai, 2013).

Acetylene, Stabilized gasmix and Propane (with normal and high speed) are some of the technologies that can be used for cutting and edge grinding of plates (Farkas and Jármai, 2013). These methods are however not really used much in South Africa, where saw and pressure (guillotine) cutting are rather used.

The plate cutting and edge grinding time (T_{CP}) can be calculated with the following equation (Farkas and Jármai, 2013):

$$T_{CP} = \sum_i C_{CPi} \cdot t_i^n \cdot L_{ci} \tag{2.13}$$

where t_i $[mm]$ is the plate thickness and L_{ci} $[mm]$ is the cutting length. The n value is obtained from curve fitting calculations (Farkas and Jármai, 2013).

2.5.4.7 Time for flattening plates

The time for flattening plates (T_{FP}) can be calculated with the following equation (Farkas and Jármai, 2013):

$$T_{FP} = \Theta_{df} \cdot \left(a_e + b_e \cdot t^3 + \frac{1}{a_e \cdot t^4}\right) \cdot A_p \tag{2.14}$$

where Θ_{df} is the difficulty parameter (Θ_{df} = 1, 2 or 3, depending on the form of the plate), a_e = $9.2 \times 10^{-4} min/mm^2$ and $b_e = 4.15 \times 10^{-7} min/mm^5$.

2.5.4.8 Surface preparation time

The surface preparation time (T_{SP}) can be calculated with the following equation (Farkas and Jármai, 2013):

$$T_{SP} = \Theta_{ds} \cdot a_{sp} \cdot A_s \tag{2.15}$$

where Θ_{ds} is the difficulty parameter and $a_{sp} = 3 \times 10^{-6} min/mm^2$.

2.5.4.9 Painting time

The painting time (T_P) can be calculated as follows (Farkas and Jármai, 2013):

$$T_P = \Theta_{dp} \cdot (a_{gc} + a_{tc}) \cdot A_s \tag{2.16}$$

where Θ_{dp} is the difficulty factor (Θ_{dp} = 1, 2 or 3 for horizontal, vertical or overhead painting), $a_{gc} = 3 \times 10^{-6} min/mm^2$ and $a_{tc} = 4.15 \times 10^{-6} min/mm^2$.

The use of sections with smaller outside area reduces paint and painting time (Bresler et al., 1960).

2.5.4.10 Times of hand cutting and machine grinding of strut ends

The times of hand cutting and machine grinding of strut ends (T_{CG}) are only considered when looking at tubular structures, which is not the case in this thesis. Thus the equation for T_{CG} was not presented in this research project (Farkas and Jármai, 2013).

2.5.4.11 Cost of intumescent painting

According to Farkas and Jármai (2013), intumescent painting is becoming more and more popular in practice, because it looks attractive and protects the structure against fire. Although popular, it is relatively expensive (Farkas and Jármai, 2013).

The cost of intumescent painting (K_{pi}) can be calculated with the following equation:

$$K_{pi} = (k_p + k_{pi}) \cdot A_p \tag{2.17}$$

where $k_p = 14\ \$/m^2$ is the cost of the specific painting, k_{pi} is the additional cost of intumescent painting and A_p is the full cover surface. The cost of the specific painting (k_p) accounts for two paint layers (ground and top coat). The cost of the additional intumescent painting (k_{pi}) depends on the thickness and the thickness depends on the required protection time. The value of k_{pi} range from 20 $\$/m^2$ to 60 $\$/m^2$, for half an hour protection to one hour protection respectively.

2.6 Optimisation algorithms

Optimisation algorithms (or automated search techniques) are normally used to obtain optimal results at a fraction of the time of which it will take for an exhaustive search. Accuracy is unfortunately sacrificed in this process. An exhaustive search will always give the exact solution, while an optimisation algorithm will give a solution close to the exact solution in most cases.

Various optimisation algorithms are available. Jármai et al. (2003) provide information on how some of these optimisation algorithms compare to each other. They applied four conceptually different optimisation algorithms on a welded I-section frame and compared them to each other. The following four optimisation algorithms were used (Jármai et al., 2003):

- Genetic algorithm
- Leap-frog method
- The method of Rosenbrock
- Differential evolution technique

The leap-frog method was found to yield the best results of the four methods, according to Jármai et al. (2003). None of these optimisation algorithms were however applied to the optimisation problem in this research project, as the problem lent itself to the methodology described in this thesis.

2.7 I-section optimisation

2.7.1 Overview of research on I-section optimisation

Goble and Fred (1971) developed an optimisation technique for unstiffened I-sections, based on unconstrained minimisation. The technique has the advantage of keeping the structural design component separate from the automated optimisation part of the program. The technique can thus be treated as a closed black-box procedure (Goble and Fred, 1971) and can be used to optimise structures according to cost and weight.

Schilling (1974) derived the optimal cross-sectional properties of I-sections and specified the area ratios (A_w/A_f) that yields the maximum bending resistance and stiffness of an I-section. Refer to Section 2.7.2.1 for the optimal area ratios of I-sections.

Chong (1976) extended the work of Schilling (1974) and showed that hybrid I-sections can lead to $10 - 13\%$ weight savings, compared to homogeneous I-sections.

Azad (1978) searched for the optimum dimensions for homogeneous I-sections, while considering the available plate thicknesses and meeting the AISC specifications. He also developed design charts to aid engineers in the design of economical I-sections.

Farkas (1984) optimised homogeneous and hybrid I-sections. He showed that hybrid I-sections can lead to $20 - 40\%$ weight savings and $17 - 30\%$ cost savings. However, if the deflection constraint is active, hybrid I-sections become uneconomical.

Abuyoune and Adeli (1987) formulated an optimisation problem for the design of unstiffened and stiffened hybrid I-sections according to the 1980 AISC specifications. The General Geometric Programming (GGP) technique was employed to determine the minimum weight solution numerically and was found to be efficient for the minimum weight optimisation of hybrid I-sections.

Dhillon and Kuo (1991) employed the same method as Abuyoune and Adeli (1987), the GGP technique, to optimise composite hybrid I-sections according to the AASHTO specifications.

Sarma and Adeli (1999) summarised the research done on cost optimisation of steel structures, which also included beams and plate girders. They also presented a general cost function for beams and plate girders (refer to Section 2.7.2.2).

Griffiths and Miles (2003) used a Generic Algorithm (GA) to find the optimum cross-section of a beam subjected to several load conditions. Griffiths and Miles (2003) made use of the process Shape Discovery to obtain an optimal cross-section by evolving an initial population of random cross-sections (including I-section shapes). Shape Discovery is the process of evolving a feasible cross-section without the use of a base cross-section. Although the process showed promising results, the research could not be applied in this thesis, as specialist shapes cannot easily be formed with steel or steel plates (Griffiths and Miles, 2003).

Xisheng and Shier (2005) presented information on how to design general twin symmetric I-sections with the optimization method ANSYS, which provides an analysis-assessment-fixed cycle. The cycle repeats until all the design requirements are satisfied.

Heinisuo et al. (2007) looked at the design of welded steel I-sections, as primary and secondary beams (refer to Section 1.4)in typical steel structures, based on the criteria of the relevant Eurocodes. Particle Swarm Optimization (PSO) was employed to solve the structural design problems at the lowest possible cost. These solutions were then compared to exact solutions, obtained using an exhaustive search. Only the ultimate limit state was considered while deflection was ignored, because when the deflection is too large, the beam can be pre-cambered, according to Heinisuo et al. (2007). I-sections with flange widths ranging from 50 to 1000 mm and web heights ranging from 80 to 1000 mm were considered, in increments of 5 mm. The plate thicknesses ranged between 4 mm and 35 mm, with steel yield stresses of 235 MPa and 355 MPa. It was found that the PSO method yielded poor results for predefined I-sections, when compared to the exact solution (PSO method typically 10 % greater). For this reason the layered PSO method was implemented. The layered PSO method yielded good results, with the error in most cases below 5 % (Heinisuo et al., 2007).

Senouci and Al-ansari (2009) optimised the cost of composite beams with the use of genetic algorithms. The model included the cost of the steel beam, shear studs and concrete. The composite beams were designed according to the AISC-LRFD specifications and the optimisation methodology yielded solutions which were 11 to 25 % cheaper than normal AISC design solutions.

Mela and Heinisuo (2014) did the most recent research on the optimisation of welded I-sections. They optimised homogeneous and hybrid I-sections and found that hybrid I-sections can be cheaper than normal S355 steel I-sections for loads larger than 60 kN/m. Hybrid I-sections with S355 steel in combination with S500 and S700 steel were tested, as well as hybrid I-sections fully fabricated with S500 and S700 steel. These tests confirmed that hybrid I-sections are the most optimal when S355 steel is used in combination with S500 and S700 steel (refer to Section 2.7.3.2).

2.7.2 General I-section optimisation

2.7.2.1 Optimal cross section of an I-section

The bending resistance (elastic or plastic) and the bending stiffness of an I-section depend on the proportions of their cross section (Schilling, 1974). The elastic bending resistance of an I-section depends on the elastic section modulus (Z_{el}) and the plastic bending resistance on the plastic section modulus (Z_{pl}). The stiffness of an I-section depends on the moment of inertia (I) (Schilling, 1974).

If the width-to-thickness ratio of the flange is small enough to ensure that local buckling will not occur, the elastic and plastic section moduli, as well as the moment of inertia, are functions of three geometric properties (Schilling, 1974):

1 the total cross-sectional area (A);

2 the area ratio (A_w/A);

3 and the web slenderness (h_w^*/t_w).

Where A_w is the web area, h_w^* is the web depth and t_w is the thickness of the web.

Schilling (1974) found the optimum values of area ratio (A_w/A) corresponding to maximum values of Z_{el}, Z_{pl}, and I to be 0.500, 0.667, and 0.750, respectively.

This means that the maximum possible elastic section modulus (Z_e) is obtained by placing half of the available material in the web. Similarly, the maximum plastic section modulus is achieved with an area ratio of 0.667 and the moment of inertia with an area ratio of 0.750 (Schilling, 1974).

It is apparent from Figure 2.4 that the area ratio (A_w/A) can vary significantly from its optimum value without greatly reducing Z_{el}, Z_{pl}, or I. The elastic section modulus (Z_{el}) is within 98% of its optimum value when the area ratio is between 0.39 and 0.62 (Schilling, 1974).

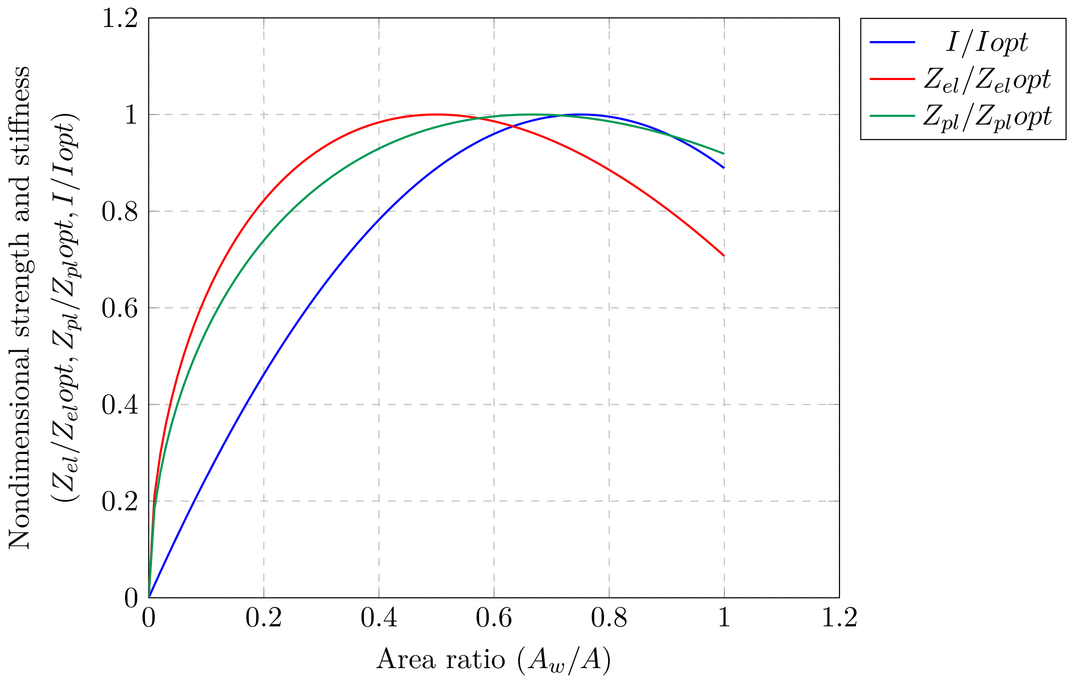


Figure 2.4: Strength and stiffness as a function of area ratio for I-beams (Schilling, 1974)

The elastic section modulus (Z_{el}) and moment of inertia (I) are both approximately 97.5% of their optimum values at area ratio (A_w/A) of 0.63, which is also close to the optimal area ratio needed to obtain the maximum plastic section modulus (Z_{pl}), $A_w/A = 0.667$ (Schilling, 1974).

Thus for an I-section to be strong in bending and relatively stiff (practical) it should have an A_w/A ratio in the region of 0.63.

2.7.2.2 Minimum cost optimisation of I-sections

Sarma and Adeli (1999) and Mela and Heinisuo (2014) presented cost functions for I-sections. Both cost functions considered all the factors considered by Farkas and Jármai (2013) (refer to Section 2.5.4), but also considered transportation and erection cost additionally. Refer to Sarma and Adeli (1999) and Mela and Heinisuo (2014) for additional information.

2.7.3 Hybrid I-section optimisation

2.7.3.1 Definition of hybrid I-sections

Developments in material technology and manufacturing processes over the years have led to the availability and use of high strength steels, with yield stresses up to 700 MPa. Such steels can be used to fabricate welded I-sections which are stronger and lighter than I-sections fabricated with S355 steel (homogeneous I-sections). If the web is made of a weaker steel than the flange, the section is called a hybrid I-section (Mela and Heinisuo, 2014).

There is also the problem that higher strength steel typically implies increased manufacturing cost, which should be considered when pursuing an economical I-section design (Mela and Heinisuo, 2014).

2.7.3.2 Minimum weight and cost optimisation

Quite a few researchers have studied the optimisation of hybrid I-sections (refer to Section 2.7.1). Mela and Heinisuo (2014) considered the optimisation of hybrid and homogeneous I-sections, for both cost and weight under the design rules of Eurocode 3. They took into account the bending and shear resistance during the optimisation process, but neglected lateral torsional buckling, assuming that sufficient support is provided to the I-sections. The I-sections were also designed and optimised under simply supported uniformly loaded conditions.

The cost function used by Mela and Heinisuo (2014) accounted for the cost of material, fabrication, transportation and erection on site.

After the optimisation process, the minimum weight solutions of the S355 steel were compared to the S500 and S700 minimum weight solutions. The comparison of the minimum weight results are shown in Table 2.3. The values in Table 2.3 are the ratios of the S500 and S700 minimum weight solutions to the S355 minimum weight solutions.

Table 2.3 shows that hybrid I-sections only become really effective for design parameters with loads larger than 60 kN/m and that it can provide weight savings ranging between 2 % and 34 %, depending on the design problem.

According to Mela and Heinisuo (2014), HSS solutions are mostly restricted by the displacement constraint where the higher strength steel does not provide any benefit. The S335 solutions, on the other hand, are mostly restricted by moment resistance.

Table 2.3: Results of Mela and Heinisuo (2014) I-section weight minimisation.

S500 Load (kN/m)	**Span (m)**			**S700 Load (kN/m)**	**Span (m)**		
	6	**8**	**10**		**6**	**8**	**10**
20	0.98	0.97	0.90	**20**	0.98	0.97	0.90
60	0.84	0.84	0.84	**60**	0.75	0.74	0.73
100	0.84	0.84	0.77	**100**	0.71	0.67	0.66

The minimum cost solutions of the S355 steel were also compared to the minimum cost solutions of S500 and S700 steel, as well as hybrid I-sections with a combination of high strength steel and S355 steel. The comparison of the minimum cost results are shown in Table 2.4 and the properties of the 14 hybrid designs are shown in Table 2.5.

Table 2.4: Results of Mela and Heinisuo (2014) cost minimisation

S500 Load (kN/m)	**Span (m)**			**S700 Load (kN/m)**	**Span (m)**			**Best Load (kN/m)**	**Span (m)**		
	6	**8**	**10**		**6**	**8**	**10**		**6**	**8**	**10**
20	1.15	1.15	1..11	**20**	1.26	1.27	1.22	**20**	1.00 (1)	1.00 (1)	0.99 (2)
60	1.06	1.03	1.01	**60**	1.11	1.06	1.03	**60**	0.96 (10)	0.93 (4)	0.90 (11)
100	1.04	1.04	0.94	**100**	1.04	0.97	0.94	**100**	0.93 (10)	0.91 (10)	0.90 (5)

Table 2.5 shows that the HSS solutions of only S500 and S700 steel are only beneficial for really high loads over large spans. Hybrid solutions with a combination of S355 steel with S500 and S700 steel, on the other hand, becomes beneficial for loads equal to and larger than 60 kN/m. Hybrid I-sections can yield cost savings of up to 10 % for large loads.

Table 2.5: Numbering of steel grade combinations (Mela and Heinisuo, 2014)

Number	Top flange	Web	Bottom flange
1	S355	S355	S355
2	S500	S355	S500
3	S500	S500	S500
4	S700	S355	S700
5	S700	S500	S700
6	S700	S700	S700
7	S355	S355	S500
8	S355	S355	S700
9	S500	S355	S355
10	S700	S355	S355
11	S500	S355	S700
12	S500	S500	S700
13	S700	S355	S500
14	S700	S500	S500

2.7.3.3 Comparison of minimum weight and minimum cost optimisation

Figure 2.5 shows the minimum weight and cost solutions of the 14 hybrid designs of Table 2.5.

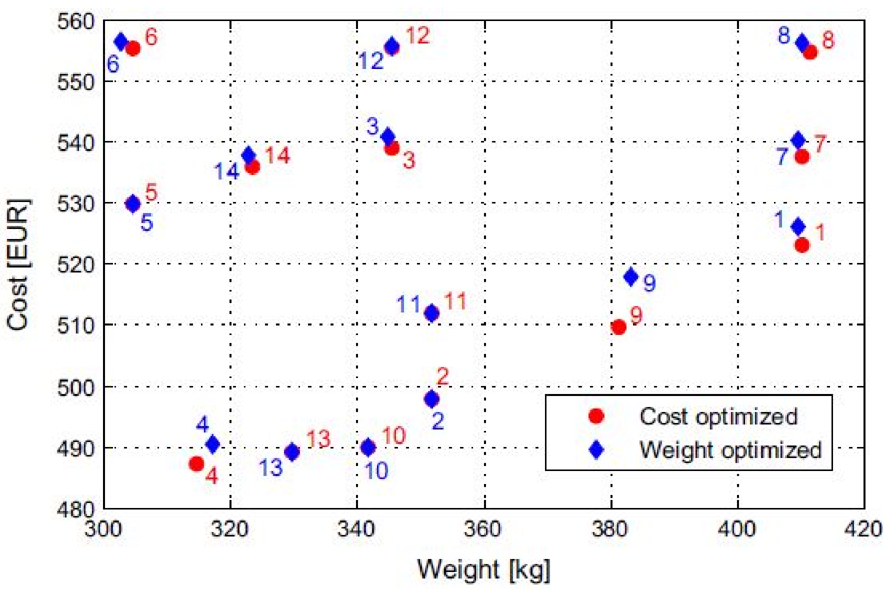


Figure 2.5: Minimum cost and minimum weight solutions of the design problem with q = 60 kN/m and L = 8 m (Mela and Heinisuo, 2014)

The minimum weight and cost solutions coincide for several of the hybrid I-sections in Figure 2.5, with the difference between the rest being virtually negligible, except for design problem 4 and 9. These differences were, however, caused by problems with the different optimisation methods.

2.7.3.4 Conclusion about hybrid I-sections

Although hybrid I-sections can provide lighter solutions for large spans and high loads, they tend to produce more expensive solutions than those for conventional S355 steel I-sections for small loads, which occur more often in practice. It would therefore be uneconomical to use hybrid I-sections in common design situations.

2.8 Conclusion

No reports on optimisation research could be traced that is directly applicable to the this research project. Nevertheless, some of the available information could be used to provide insight or help solve parts of the research problem.

Some research (refer to Section 2.7.1) have found that large weight savings can be obtained by using welded I-sections instead of hot-rolled I-sections, but hot-rolled I-sections were always cheaper in the past, due to the larger fabrication cost of welded I-sections, the consequences of lack in technology. This has however changed and the fabrication cost of welded I-sections has decreased over the years with the advancement of technology. The cost of hot-rolled I-sections has also increased over the years in South Africa, due to extra import costs and time delays. A set of welded I-section could therefore potentially be cheaper than the set of hot-rolled sections available in South Africa.

This chapter confirmed that minimum weight optimisation of the welded I-sections will be sufficient to achieve the aim of this project, as Mela and Heinisuo (2014) found that the solution produced by minimum cost optimisation is not much different from the solution obtained with minimum weight optimisation for welded I-sections (refer to Section 2.7.3.2). In addition, cost optimisation can easily produce wrong results with the wrong information and a great deal of research is required to produce accurate cost functions (refer to Section 2.5.4).

Chapter 3

Welded I-section requirements

3.1 Introduction

Welded I-sections have to meet certain design and fabrication requirements. Some of these requirements overlap, as explained in this chapter.

Designer requirements are as follows:

- sufficient strength (capacity)
- ease of attachment
- sufficient corrosion resistance
- adequate availability
- reasonable cost

For the welded I-sections to be fabricated at the least possible cost, designers need to ensure that the I-sections also meet the main requirement of steel fabricators, being ease of fabrication (constructability). In addition, welded I-sections should also be easy to handle in the workshop and on site.

3.2 I-section capacity

For welded I-sections to have sufficient capacity and strength, they must have the following properties (Blodgett, 1966):

- Sufficient moment resistance, measured by section modulus (Z)
- Sufficient stiffness, measured by moment of inertia (I)
- Sufficient shear resistance, measured by web area (A_w)
- Ability to withstand web buckling, which is ruled by the web slenderness ratio (h_w^*/t_w).
- Sufficient weld capacity

SANS 10162-1 (SABS, 2011c) should be used to calculate the moment, shear and weld capacity of I-sections, as this is the code used in South Africa (refer to Section 6.3). Where SANS 10162-1 has shortcomings, the Canadian steel design code, S16-14 (CSA Group, 2014), is preferred, as demonstrated in Section 6.3. The loading code, SANS 10160-2 (SABS, 2011b), must be used to determine the applied loads on structures.

The deflection of the welded I-sections should be checked in order not to exceed a value of $span/300$, in accordance with the deflection limit specified by SANS 10162-1 (SABS, 2011c) for members supporting floors.

3.3 Welding

The following issues are relevant when specifying welding:

- required capacity of welds
- weldability of steel
- welding process used
- minimum size of weld
- fatigue resistance of welded joints

3.3.1 Weld capacity

The welds connecting the plates should always have sufficient strength over the entire length of the I-section to withstand any longitudinal shear resulting from a change in moment over the entire length of the I-section.

The calculations from SANS 10162-1 (SABS, 2011c) will specify a weld size that will have sufficient capacity to withstand the longitudinal shear (refer to calculations in Section 3.3.1).

The shear resistance of a weld is based on the weld size, weld metal ultimate tensile strength and the angle of the weld. The weld angle is always equal to 0^o on welded I-sections and the ultimate tensile strength is always going to be 480 MPa, according to SANS 10162-1 (SABS, 2011c), as only S355JR steel is used in this research project. With this information the required weld size can be calculated. There is however a limit on the minimum weld size, according to the thickness of the plate it is applied to, as presented in Section 3.3.6.

With the use of SANS 10162-1 (SABS, 2011c), it was found that the maximum weld size that can be used on welded I-sections, without wasting material and without accounting for fabricating limitations, is 0.7 times the minimum plate thickness between the web and flange. The ratio of 0.7 was derived with the assumption that the shear in the smaller plate is equal to the shear in the two welds, which will be the case when the I-section reaches its maximum shear resistance. The ratio was derived as follows:

Web shear resistance (refer to Section 13.4 of SANS 10162-1 (SABS, 2011c)):

$$\begin{aligned} V_r &= \phi \cdot A_v \cdot f_s \\ &= 0.9 \cdot A_v \cdot 0.66 \cdot f_y \\ &= 0.9 \cdot t_w \cdot l \cdot 0.66 \cdot f_y \end{aligned} \tag{3.1}$$

where A_v is the shear area, f_s is the ultimate shear stress, f_y is the yield stress (355 MPa), t_w is the web thickness and l is the web length.

Weld shear resistance (refer to Section 13.13.2 of SANS 10162-1 (SABS, 2011c)):

$$\begin{aligned} V_r &= 0.67 \cdot \phi w \cdot A_{ww} \cdot x_u \cdot (1 + 0.5 \cdot sin^{1.5}\theta) \\ &= 0.67^2 \cdot 2 \cdot a_v \cdot cos(45^o) \cdot l \cdot x_u \\ &= 0.67^2 \cdot 2 \cdot a_v \cdot 0.707 \cdot l \cdot x_u \end{aligned} \tag{3.2}$$

where A_{ww} is the weld area, x_u is the minimum weld ultimate tensile strength (480 MPa), a_v is the weld thickness and θ is the weld angle (0^o).

Web shear resistance = Weld shear resistance:

$$\begin{aligned} 0.9 \cdot t_w \cdot l \cdot 0.66 \cdot f_y &= 0.67^2 \cdot 2 \cdot a_v \cdot 0.707 \cdot l \cdot x_u \\ 0.9 \cdot t_w \cdot l \cdot 0.66 \cdot 355 &= 0.67^2 \cdot 2 \cdot a_v \cdot 0.707 \cdot l \cdot 480 \\ a_v &= 0.69 \cdot t_w \\ a_v &\approx 0.7 \cdot t_w \end{aligned} \tag{3.3}$$

3.3.2 Weldability of steel

The term weldability of steel refers to the ease of creating an acceptable, crack-free, complete joint (weld) (Blodgett, 1966).

Steel sold for structural purposes in South Africa are all weldable and can be welded without difficulty. Some steels are however more suitable for high-speed welding than others. Electrode core wire is usually held to high standards to produce good welds, but because plate metal becomes part of the weld, plates of low standard can produce inconsistent welds. More of the plate metal is also mixed with the weld at high welding speeds, because of the high currents used to obtain high welding speeds. Quality control on the plate analysis is thus important to ensure that the welds are correctly designed by engineers (Blodgett, 1966).

According to the Red Book (SAISC, 2013), steel has to have a carbon equivalent (CE) less than 0.51 % to be regarded weldable. The CE value can be calculated as follows (SAISC, 2013):

$$CE = C + \frac{Mn}{6} + \frac{Cr + Mo + V}{5} + \frac{Ni + Cu}{15} \tag{3.4}$$

where C, Mn, Cr, Mo, V, Ni and Cu is, respectively, the percentage of carbon, manganese, chromium, molybdenum, vanadium, nickel and copper in the steel. The carbon content of all steel produced in South Africa can be assumed not to exceed 0.3 % according to SAISC (2013).

With steel that is easy to weld, complicated welding procedures or expensive electrodes are generally unnecessary (Blodgett, 1966).

3.3.3 Welding processes

The following four welding processes are often used in South Africa (De Clercq, 2012):

- Shielded metal arc welding (SMAW)
- Submerged arc welding (SAW)
- Gas metal arc welding (GMAW)
- Flux cored arc welding (FCAW)

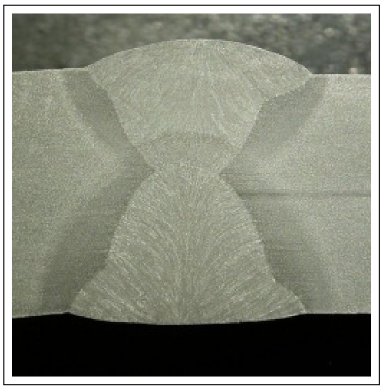
Each process has its own advantages and disadvantages, defined according to speed, weld positions, etc. All of these advantages and disadvantages should be accounted for when fabricating welded I-sections in mass. Guidance in this regard can be obtained from the Green Book (De Clercq, 2012).

3.3.4 Types of welded connections

There are a number of basic types of welds and joints. However, only one joint type applies to welded I-sections, namely tee joints and normally only one type of weld, namely fillet welds (shown in Figure 3.1a). Groove welds can also be used on welded I-sections occasionally (refer to Figure 3.1b).

Fillet welds are characterised by its triangular cross section that commonly has equal legs. Slot and plug welds are used when sufficient fillet welds cannot be secured (Blodgett, 1966).

(a) Fillet weld



(b) Groove weld

Figure 3.1: Different types of welded connections (Steelconstruction.info, 2016b)

3.3.5 Fatigue resistance of welded connections

Welds fail because of fatigue failure when they are subjected to repeated loads, as is normally encountered with crane girders. To reduce the occurrence of fatigue problems quality standards must be employed, especially when referring to surface defects and cracks. Refer to the Green Book (De Clercq, 2012) for more information.

3.3.6 Minimum size of welds without preheating

It is advisable to place a limit on the weld size according to the thickness of the thicker part joined. This limit is advised mainly to avoid cracking of fillet welds. The cracking is caused by the rapid dissipation of heat into the parent metal (De Clercq, 2012).

The limits laid down by the Green Book (De Clercq, 2012) are shown in Table 3.1. The weld size should not exceed the thickness of the thinner plate in the joint (De Clercq, 2012).

Table 3.1: Minimum size of fillet welds without preheating (De Clercq, 2012)

Thickness of thicker part (mm)	**Minimum fillet weld size (mm)**
Up to 12	5
Over 12 to 20	6
Over 20 to 40	8
Over 40 to 60	10
Over 60 to 150	12

If smaller welds need to be used than specified in Table 3.1, preheating should be specified. However, preheating should only be specified when it is really needed, as it is expensive according to Blodgett (1966). Some guidance can be obtained from the Green Book (De Clercq, 2012).

3.4 Ease of attachment

Designers require welded I-sections to be easily attachable to other sections. An I-section will not be used frequently when it is difficult to attach to other structural members, to form the desired structures. This requirement goes hand-in-hand with the requirement of steel fabricators, that I-section flange widths should be equal to or larger than 100 mm, as it is difficult to drill holes in flanges smaller than 100 mm (refer to Appendix B). Engineers will sometimes use columns with larger flange widths to create more space for attachments.

The Green Book (De Clercq, 2012) contains all the standard steel connections used in South Africa. Although all the standard steel connections are available, it is difficult to take these connections into account when searching for an optimal set of welded I-sections, as the connection will be different for each different design parameter.

3.5 Constructability of welded I-sections

Steel fabricators have two requirements for welded I-sections for it to be fabricatable. The cross section's dimensions should firstly correspond to the available steel plate sizes, thicknesses and widths. Secondly, the plates should be easy to weld without problems, like distortion.

3.5.1 Available steel plates

Several different forms of steel plate are available in South Africa. All of the plates can be classified as either flat bars, hot-rolled sheets or standard steel plates. All of these plate types can be used to fabricate welded I-sections, but each has its own advantages and disadvantages. More information regarding the available steel plates can be obtained from Section 4.3.1.

3.5.2 Ease of welding

According to Azad (1978), plate thicknesses less than 6 mm are hardly ever considered in the practical design of built-up members. However, the interviewed steel fabricator considers 5 to 6 mm plates to be weldable, as weld technology has improved over the years (refer to Appendix B).

According to the steel fabricator interviewed, the smallest weldable plate is 4 mm thick. A 4 mm plate is however difficult to weld and is regarded unpractical for the fabrication of welded I-sections, according to Heinisuo et al. (2007). A 5 mm plate is typically the lowest plate thickness which can be used to fabricate welded I-sections (refer to Appendix B).

According to the interviewed steel fabricator, the welding process (method) has a big influence on the temperatures reached in the plate, which in turn influences the distortion of the plates (refer to Appendix B). Submerged arc welding (SAW), for example exposes the welded plate to higher temperatures than normal fillet welding by hand (SMAW).

Fillet welding by hand is usually used for thinner plates, according to the interviewed steel fabricator (refer to Appendix B), as they do not expose the welded plate to very high temperatures, as is the case with submerged arc welding. Gas metal arc welding (GMAW) is one of the welding processes usually conducted by hand, but is adaptable to automated welding. The smaller the temperature of the weld, the smaller the chance of distortion of the plate.

Submerged welding is used on thicker plates and is usually done by automated machinery. Submerged welding is faster than fillet welding, but can only be used on thicker plates, as it generates intense volumes of heat (refer to Appendix B). Thicker plates can absorb more heat without distorting.

3.6 Plate Nesting

Nesting refers to maximising the percentage of the steel purchased that is actually used in the structures, which amounts to minimising waste. As welded I-sections consist of long plates, the challenge is to use as much of the width of a plate in the final product.

3.7 Manufacturing equipment

The suitability and adequacy of manufacturing equipment can make a big difference in the cost of welded I-sections. Similarly the cost of such equipment should be accounted for in the cost of the welded I-sections over the long term.

A special plant may be needed in the future in order to fabricate welded I-sections at a reasonable cost.

3.8 Corrosion resistance

Hot-rolled I-sections are normally protected by either paint coating or hot-dip galvanising (metallic coating). Welded I-sections should, however, preferably be protected by paint, as they are difficult to galvanize according to the CEO of Union Steel (refer to Appendix B). Paint coating is also the most commonly used method of protecting steel (Steelconstruction.info, 2015b).

The galvanizing proses (shown in Figure 3.2) can easily cause distortion of the webs and flanges of welded I-sections, because the steel would be heated to temperatures above $450^{o}C$; causing residual stresses in the section to dissipate. The beam and its elements, straight under the initial conditions of residual stress, can distort when the residual stresses disappear.

With relatively small width-to-thickness ratios the distortion tends to be less of a problem. Straightening after galvanising is an option to be considered.

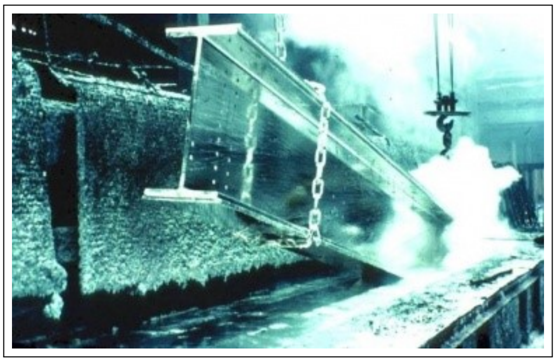


Figure 3.2: Hot-rolled I-section withdrawn from a typical hot-dip galvanizing bath (Steelconstruction.info, 2015a)

3.9 Ease of handling in workshop and on site

As mentioned by the CEO of Union Steel (Steel fabricator), handleability in the workshop is not really a problem for any steel fabricator, as it is a job they know well (refer to Appendix B).

Union Steel normally handles I-sections with lengths up to a maximum of 13 m without difficulty and can handle beams up to 24 m. Handleability is a bigger problem on site, as the maximum beam span that can fit onto a normal size truck is 13 m. Furthermore the maximum mass of a member should normally be less than 6 tons, to enable erection on typical projects (De Clercq, 2010).

3.10 Fabrication defects and applicable standards

Steel fabricators need to ensure that their product (welded I-sections) is of adequate quality, in order to achieve the required capacity and to meet other requirements (refer to Section 3.1). Fabrication defects are one of the parameters which should be controlled to ensure that welded I-sections are of sufficient quality.

According to Bresler et al. (1960), the following fabrication defects should be accounted for in the properties of welded I-sections, to ensure that the cost of welded I-sections is not unnecessarily increased; namely:

- Dimensional tolerances and accuracy of the various I-section parts. If the tolerances of welded I-sections are too rigid the cost will unnecessarily be increased.
- Straightness of large members. It is not possible to keep large members precisely straight, but the deviations can be kept within reasonable limits and not affect structural usefulness.
- Straightening plates and I-sections. Straightening can cause residual stresses in members, which can lead to distortion under certain conditions.

Fabricating steelwork in accordance with the requirements of SANS 2001-CS: 2005 (SABS, 2005) will ensure that defects are within adequate tolerances, to ensure that the cost of welded I-sections is not unnecessarily increased by excessively high standards.

SANS 2001-CS: 2005 (SABS, 2005) specifies the quality of processes and products; and Table 5 of the code provides the permissible deviations of welded I-sections, which should be maintained by steel fabricators.

Welding electrodes and other welding consumables used on welded I-sections should also comply with the requirements of AWS D 1.1 (AWS, 2004). This standard is supported by years of research and actual testing (Blodgett, 1966).

3.11 Weld inspection and quality

Weld inspection and quality control are required after and during the fabrication proses of welded I-sections.

The weld quality can be determined by tests or inspection. The inspection of welds is normally performed by the steel fabricator, the designer and specialised inspectors through visual inspection. According to Blodgett (1966), steel fabricators and designers can apply the following points to obtain welds at the least possible cost, while still meeting the actual service requirements:

- Proper design of joints and connections
- Good welding procedure
- Good welding workmanship and techniques
- Responsible, intelligent inspection

It was found by Blodgett (1966) that a weld under steady tensile load which exhibits a modest amount of one of the following defects: undersize, undercut, porosity, or lack of fusion, considered individually, is stronger than the plate it is connected to. However, if the weld exhibits a combination of these defects, it could become weaker than the plate it is connected to and for this reason a predetermined percentage of the welded I-sections produced by fabricators should be inspected to ensure that they are of the required quality. Blodgett (1966) also confirms that excessive precautions to obtain high quality welds, beyond the required service requirements, are unnecessary, expensive and serves no practical purpose (Blodgett, 1966).

It is difficult to take weld defects into account in the design of welded I-sections. The quality of welds was therefore not accounted for in this research project, on the assumption that the fabricator will produce welding of adequate strength.

More information about weld defects, quality control and methods of inspections can be obtained from the Chapter 4 of the Green Book (De Clercq, 2012) and SANS 2001-CS1: 2005 (SABS, 2005).

3.12 Stock keeping

Designers require steel elements to be readily available, in order for them to specify an element with confidence that delays will not occur. Sufficient quantities of welded I-sections should therefore be kept in stock and be readily available for engineers to use. With welded I-sections, the stock keeping can be reduced to keeping only steel plates in stock instead of bulky I-sections. I-sections are then only manufactured as required. Less space will thus be needed for storage of welded I-sections in comparison to hot-rolled I-sections.

3.13 Conclusion

As shown in this chapter, there is a substantial number of requirements that have to be met when designing and fabricating welded I-sections. Similarly, most of these requirements must be accounted for in the development of an economical set of welded I-sections.

The optimal set developed in this thesis could not be optimised according to cost, due to the complexity of cost functions (refer to Section 2.5) and time constraints. Accordingly, only weight and capacity requirements were used in this research project.

Chapter 4

Creation of initial I-sections

4.1 Introduction

Initial sets of welded I-sections were created following the methodology presented in this chapter. These initial sets were then used in the methodology presented in Chapter 5 to obtain different optimal sets. From these optimal sets the best practical set of welded I-sections was chosen.

The first initial set of welded I-sections, Initial Set 1, was created to be similar to the entire set of hot-rolled I-sections available globally (see Section 4.7). The other initial sets, Initial Set 2, 3 and 4, were created using practical considerations and different dimension increments (see Section 4.8).

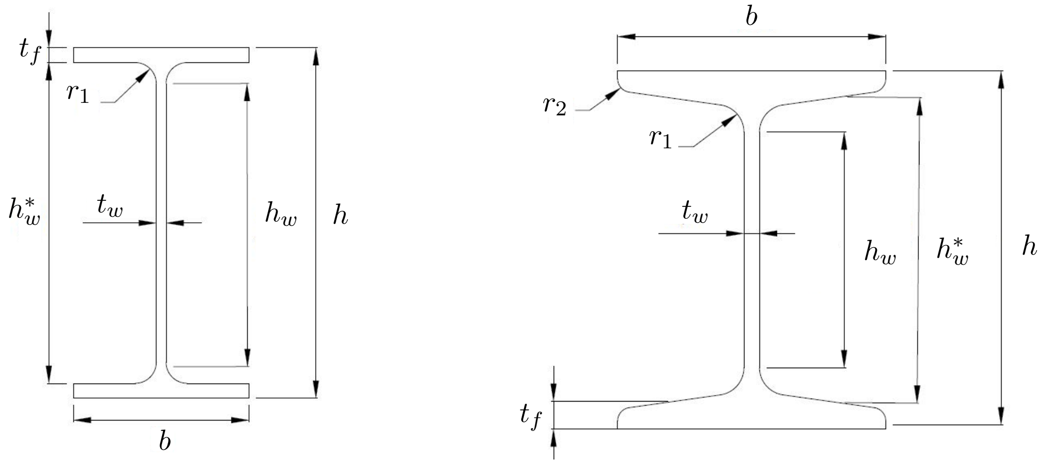
4.2 I-section dimensions

The dimensions of an I-section determine its cross-sectional shape and properties, which in return determine the strength and stiffness of a section, assuming the yield stress remains constant.

Hot-rolled I-sections can be categorised into two groups, namely parallel flange I-sections and taper flange I-sections. These groups have different cross-sections, which are determined by the different section dimensions applicable, as shown in Figure 4.1.

In Figure 4.1 h is the section depth, h_w^* is the inner depth between flanges (web depth), h_w is the depth of the straight potion of the web, b is the flange width, t_f is the flange thickness, t_w is the web thickness, r and r_1 is the radius of root fillet and r_2 is the toe radius.

The dimensions of prismatic welded I-sections are similar to that of hot-rolled parallel flange I-sections. The only difference between the two is that no fillet radius (r) is accounted for when designing welded I-sections. The dimensions of welded I-sections are shown on Figure 4.2.



(a) Parallel flange I-section

(b) Taper flange I-section

Figure 4.1: Definition of hot-rolled I-section dimensions

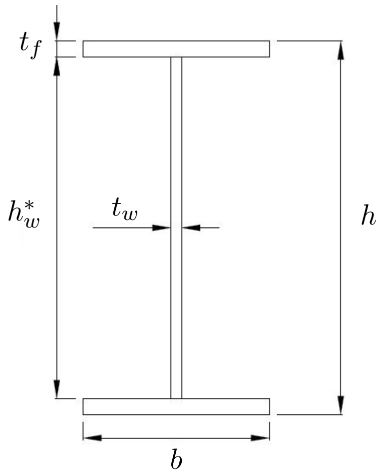


Figure 4.2: Definition of welded I-section dimensions

The following equations (Equation 4.1 and 4.2) can be used to calculate h_w and h_w^* of parallel flange H and I-sections.

$$h_w = h - 2 \cdot t_f - 2 \cdot r \tag{4.1}$$

$$h_w^* = h - 2 \cdot t_f \tag{4.2}$$

4.3 Practical considerations for welded I-sections

There are a number of practical considerations which should be considered when creating welded I-sections, to meet the requirements of steel fabricators and designers (refer to Chapter 3). These practical considerations were obtained from the following references:

- Steel fabricator, CEO of Union Steel (refer to Appendix B)
- SANS 10162-1 (SABS, 2011c)
- The Red Book (SAISC, 2013)

4.3.1 Available plate materials

4.3.1.1 Plate materials in general

As mentioned in Chapter 3, the available steel plate sizes and thicknesses have to be considered when fabricating welded I-sections in South Africa, as they have an influence on the optimal dimensions of welded I-sections. The web depth and flange width sizes also have a great influence on plate nesting (material waste).

The available plate dimensions were obtained from the Red Book (SAISC, 2013) and verified by means of Macsteel's (MACSTEEL, 2015) and Trident Steel's websites (AVENG Trident Steel, 2016).

There are many different standard plate sizes available according to the Red Book (SAISC, 2013), but in reality they are not always readily available.

There are three types of steel plate to choose from when fabricating welded I-sections, namely: flat bars, hot-rolled sheets and standard steel plates.

4.3.1.2 Flat bars

Flat bars are convenient to use for flanges and webs, subject to availability in the required sizes, as no plate cutting is required when using flat bars. Table 4.1 shows the dimensions and properties of standard flat bars available in South Africa.

Flat bars are available in standard lengths of 4 m to 13 m in 1 m increments and flat bars with widths up to 90 mm are typically only available in Commercial Quality steel (SAISC, 2013). As only S355JR steel was considered in this research project (refer to Section 1.4), flat bars up to 90 mm were not considered.

Table 4.1: Dimensions and properties of flat bars (SAISC, 2013)

Thickness (mm)	**Mass in kg/m for widths in mm**									
	20	**25**	**30**	**40**	**45**	**50**	**60**	**65**	**70**	**80**
5	0.785	0.981	1.18	1.57	1.77	1.96				
6	0.942	1.18	1.41	1.88	2.12	2.36	2.83	3.06	3.30	3.77
8	1.26	1.57	1.88	2.51		3.14	3.77	4.08	4.40	5.07
10	1.57	1.96	2.36	3.14		3.93	4.71	5.10	5.50	6.28
12		2.36	2.83	3.77	4.24	4.71	5.65	6.12	6.59	7.54
16				5.02		6.28	7.54	8.16	8.79	10.00
20				6.28		7.85	9.42	10.20	11.00	12.60
25				7.85		9.81	11.80	12.80	13.70	15.70
30										18.80
40										25.10

Thickness (mm)	**Mass in kg/m for widths in mm**								
	90	**100**	**110**	**130**	**150**	**180**	**200**	**250**	**300**
6	4.24	4.71	5.18						
8	5.65	6.28	6.91	8.16	9.42				
10	7.07	7.85	8.64	10.20	11.80	14.10	15.70		
12	8.48	9.42	10.40	12.20	14.10	17.00	18.80		
16	11.30	12.60		16.30	18.80	22.60	25.10	31.40	37.70
20	14.10	15.70		20.40	23.60	28.30	31.40	39.20	47.10
25	17.70	19.60		25.50	29.40	35.30	39.20	49.10	58.90
30		23.60		31.60	35.30	42.40	47.10	58.90	70.60
40		31.40			47.10	56.50	62.80	78.50	94.20
45		39.20							106.00
50									118.00

Union Steel normally uses flat bars with widths of 130 mm, 150 mm, 180 mm and 200 mm. The CEO of Union Steel also indicated that flat bars with a width of 250 mm can also be used to fabricate welded I-sections, but are not often used (refer to Appendix B). All of the mentioned flat bars are available in S355JR steel.

Flat bars with widths of 110 mm were not considered in this research project, as they are not readily available according to the CEO of Union Steel (refer to Appendix B).

4.3.1.3 Hot-rolled sheets

The benefit of hot-rolled sheets, available as coils, is that the section span is not limited by the plate size, as is the case with flat bars and standard steel plates.

Hot-rolled sheets do however have drawbacks, the first being that sheets have to be straightened when used to fabricate welded I-sections, requiring expensive equipment. For small scale I-section fabrication, standard plates and flat bars will therefore be more economical. However, if welded I-sections are produced in mass by the application of the required equipment, sheets could become economically viable.

The second drawback is that hot-rolled sheets are only available in small thicknesses (refer to Table 4.2) and are therefore mostly used to fabricate the webs of welded I-sections.

Table 4.2: Standard dimensions of hot-rolled sheets (SAISC, 2013)

Standard thicknesses (mm)		
1.2 1.4 1.5 1.6 1.8 2 2.2 2.5 2.8 3 3.5 4.5 5 6 8 10 12		
Standard widths		
Thickness range (mm)	Widths (mm)	Increments (mm)
t <2.5	$600 \leq b \leq 1500$	25
t = 2.5	$600 \leq b \leq 1500$	25
$2.8 \leq t \leq 12$	$600 \leq b \leq 1800$	25
Standard lengths		
1000 to 6000 mm in increments of 25 mm, subject to:		
Thickness $\leq$ 1.8 mm: maximum length = 3650 mm		
Width >1225 mm: minimum length = 1800 mm		

According to the CEO of Union Steel (refer to Appendix B), hot-rolled sheets can have up to 2 m widths, limited to a thickness of 12 mm. There are thus wider hot-rolled sheets available in South Africa than what is specified in the Red Book (SAISC, 2013).

4.3.1.4 Standard steel plates

Steel plates of standard dimensions will probably be the most popular option for welded I-sections, because they are readily available and can be used to fabricate any web or flange size. The steel plates available in South Africa are presented in Table 4.3.

Union Steel always stocks 2.4 m wide plates (refer to Appendix B). This is a popular width and therefore a good plate size to consider during plate nesting.

The mass of standard steel plates is 7.85 kg/m^2 per mm thickness according to SAISC (2013) and was used to calculate the unit weight of welded I-sections.

Table 4.3: Standard dimensions of plates (SAISC, 2013)

Standard thicknesses (mm):	4.5	5	6	8	10	12	14	16	18	20
	22	25	28	30	32	35	38	40	45	50
	55	60	65	70	75	80	85	90	100	110
	115	120	125	130	140	150				
Typical sizes (mm):	2500 x 1200, 3000 x 1500, 4000 x 2000, 8000 x 2000, 10 000 x 2400, 13 000 x 2400 and 12 000 x 3000, but not all thicknesses in all sizes									

4.3.2 Plate nesting

Plate nesting is an economical consideration required by steel fabricators in order to minimise material waste, as mentioned in Chapter 3. Plate nesting should be accounted for when choosing web depths and flange widths, including the most economical plate type and size for fabrication purposes.

According to the interviewed steel fabricator, 20 mm has to be removed on each side of a plate before flanges or webs are cut from it, to remove any imperfections on edges of the plate (refer to Appendix B). This should always be accounted for when determining the number of flanges or webs which can be cut out of a steel plate. The 20 mm cut-offs were accounted for in the plate nesting procedure.

4.3.3 Weldability of plates

Although a 5 mm plate thickness is the practical limit for weldability, as mentioned in Chapter 3, it was decided to consider all plates thicker or equal to 4 mm when creating welded I-sections, because a 4 mm thick plate is the minimum plate size which is weldable (refer to Section 3.5.2).

4.4 Section dimension limits

4.4.1 Flange and web thickness limits

As mentioned above, available plate sizes (refer to Section 4.3.1) will determine the flange and web thicknesses of welded I-sections.

Although any flange thickness can in principle be matched with any web thickness, it is not practical to do so. For this purpose a study was conducted on 901 hot-rolled I-sections available globally (as per ArcelorMittal's website (ArcelorMittal, 2016)) in order to obtain insight into the limits that can be used to create more practical I-sections. The range of web to flange thickness ratios (t_w/t_f) used globally was determined, including how frequently they occur.

It was found that the web to flange thickness ratios of the sections ranged from 0.51 to 1.34, of which 98.78 % had ratios smaller than 1 and 1.22 % ratios larger than 1. These sections had an average web to flange thickness ratio of 0.66.

The flange thickness did not have any limits other than the available plate thicknesses. The web thickness did however have maximum and minimum limits.

4.4.1.1 Maximum web thickness

After considering the available information, it was decided not to include possible cross sections with a web thickness larger than the flange thickness in the initial sets, as only 1.22 % of the globally available hot-rolled I-sections have web to flange thickness ratios larger than 1. The maximum web thickness ($(t_w)_{max}$) was thus equal to the flange thickness:

$$(t_w)_{max} = t_f \tag{4.3}$$

4.4.1.2 Minimum web thickness

The minimum web thickness ($(t_w)_{min}$) was linked to the flange thickness and calculated with the use of Equation 4.4.

$$(t_w)_{min} = \frac{t_f}{6} \tag{4.4}$$

Equation 4.4 ensured that more I-sections were created and considered than what is practically needed, as the web to flange thickness ratio of 1/6 (0.17) is a lot smaller than what is found globally (0.51) and what is typically fabricated in South Africa (0.27). The plate girders are fabricated with a minimum web to flange thickness ratio of 0.27 in South Africa (SAISC, 2013).

4.4.2 Flange width

4.4.2.1 Minimum flange width

A minimum flange width (b_{min}) of 100 mm was used in the creation of welded I-sections, as it is difficult to drill holes in flanges narrower than 100 mm, which makes them unpractical (refer to Chapter 3).

4.4.2.2 Maximum flange width

No Class 4 flanges were considered in this research project. According to SANS 10162-1 (SABS, 2011c), material is actually wasted when Class 4 flanges are used, making Class 4 flanges uneconomical.

Because no Class 4 flanges were considered in this research project, the maximum flange width could be derived from the slenderness limit of a Class 3 flange and directly linked to the flange thickness. The Class 3 limit was obtained from SANS 10162-1 (SABS, 2011c).

Although the the maximum flange width could be calculated, it was also limited to the web depth, because I-sections with b larger than h_w^* are not commonly used in practice. The maximum flange width (b_{max}) could thus be calculated with Equation 4.5.

$$b_{max} = min\left(\frac{400 \cdot t_f}{\sqrt{f_{yf}}}, h_w^*\right) \tag{4.5}$$

4.4.3 Web depth

4.4.3.1 Minimum web depth

For this research project a minimum web depth ($(h_w^*)_{min}$) of 150 mm was used for I-sections used as beams and 250 mm for those used as girders. These depths correspond closely to the minimum depths of the commonly used I-beams and girders in South Africa (refer to Section 7.3).

4.4.3.2 Maximum web depth

A maximum web depth ($(h_w^*)_{max}$) of 1100 mm was used for I-sections used as beams in this research project, because the largest hot-rolled I-section available globally has a section depth of 1100 mm (refer to ArcelorMittal (2016)).

A maximum web depth ($(h_w^*)_{max}$) of 2150 mm was used for I-sections used as girders, because the largest standard available plate girder used in South Africa has a section depth of 2150 mm (refer to SAISC (2013)).

4.4.4 Web slenderness

A maximum web slenderness (t_w/h_w^*) of $83000/f_{yw}$ was considered in this research project, as specified in Section 14.3 of SANS 10162-1 (SABS, 2011c).

4.5 I-section properties

The properties of the existing hot-rolled I-sections were obtained from ArcelorMittal's website (ArcelorMittal, 2016). The hot-rolled property equations are therefore not presented in this thesis, as it can be obtained from ArcelorMittal's website (ArcelorMittal, 2016).

The following equations were used to calculate the properties of welded I-sections.

Area of section (A):

$$A = 2 \cdot t_f \cdot b + (h - 2 \cdot t_f) \cdot t_w \tag{4.6}$$

Refer to Section 4.2 for definitions of the variables.

Mass per unit length (m):

$$m = A \cdot \rho_e \tag{4.7}$$

where ρ_e is the unit mass of steel (7.85 kg/m^3).

Moment of inertia (I):

$$I_x = \frac{1}{12} \cdot \left[b \cdot h^3 - (b - t_w) \cdot (h - 2 \cdot t_f)^3 \right] \tag{4.8}$$

$$I_y = \frac{1}{12} \cdot \left[2 \cdot t_f \cdot b^3 + (h - 2 \cdot t_f) \cdot t_w^3 \right] \tag{4.9}$$

where I_x is the moment of inertia about the x-axis and I_y is the moment of inertia about the y-axis.

Elastic section modulus (Z_{el}):

$$Z_{el,x} = \frac{2 \cdot I_x}{h} \tag{4.10}$$

$$Z_{el,y} = \frac{2 \cdot I_y}{b} \tag{4.11}$$

where $Z_{el,x}$ is the elastic section modulus about the x-axis and $Z_{el,y}$ is the elastic section modulus about the y-axis.

Plastic section modulus (Z_{pl}):

$$Z_{pl,x} = \frac{t_w \cdot h^2}{4} + (b - t_w) \cdot (h - t_f) \cdot t_f \tag{4.12}$$

$$Z_{pl,y} = \frac{b^2 \cdot t_f}{2} + \frac{h - 2 \cdot t_f}{4} \cdot t_w^2 \tag{4.13}$$

where $Z_{pl,x}$ is the plastic section modulus about the x-axis and $Z_{pl,y}$ is the plastic section modulus about the y-axis.

Radius of gyration (r):

$$r_x = \sqrt{\frac{I_x}{A}} \tag{4.14}$$

$$r_y = \sqrt{\frac{I_y}{A}} \tag{4.15}$$

where r_x is the radius of gyration about the x-axis and r_y is the radius of gyration about the y-axis.

Torsion constant (J):

$$J = \frac{2}{3} \cdot (b - 0.63 \cdot t_f) \cdot t_f^3 + \frac{1}{3} \cdot (h - 2 \cdot t_f) \cdot t_w^3 + 0.29 \cdot \left(\frac{t_w}{t_f}\right) \cdot \left[\frac{\left(\frac{t_w}{2}\right)^2 + t_f^2}{t_f}\right]^4 \tag{4.16}$$

Warping constant (C_w):

$$C_w = \frac{t_f \cdot b^3}{24} \times (h - t_f)^2 \tag{4.17}$$

4.6 Material properties

The yield stress (f_y) of S355JR steel, the steel used to fabricate welded I-sections (refer to Section 1.4), is directly linked to the plate thickness and can be found in Table 4.4.

The ultimate tensile strength (f_u) of S355JR steel range from 470 to 630 MPa, according to SAISC (2013). A ultimate tensile strength value of 470 MPa was thus assumed for this research project.

An elastic modulus (E) of 200×10^3 MPa and a shear modulus (G) of 77×10^3 MPa was assumed for steel, according to SAISC (2013).

Table 4.4: Yield stress f_y (minimum) for thickness t (mm) (SAISC, 2013; European Standard, 2004)

Plate thickness ranges (mm)	**Yield stress (MPa)**
$3 < t \leq 16$	355
$16 < t \leq 40$	345
$40 < t \leq 63$	335
$63 < t \leq 80$	325
$80 < t \leq 100$	315
$100 < t \leq 150$	295

4.7 Initial Set 1

For investigation purposes, this initial set of welded I-sections was created by approximating the standard hot-rolled I-sections available globally, including those from Europe, America, Russia, Japan and South Africa. The available plate dimensions and plate nesting were taken into account in the creation of the initial set of welded I-sections.

4.7.1 Hot-rolled I-sections available globally

A large number of different hot-rolled I-sections are globally available. Some 901 of these I-sections were compared to welded I-sections in this research project. This led to the creation of 692 welded I-sections to approximate the 901 hot-rolled I-sections.

The 901 I-sections were obtained from ArcelorMittal's website (ArcelorMittal, 2016) and is presented in this section with the information of the hot-rolled I-sections used in South Africa. The information on South African I-sections was obtained from Table 2.9 to 2.11 of the Red Book (SAISC, 2013).

Each country has its own set(s) of I-sections. Some countries, like South Africa, use I-sections that originated in other countries. The following subsections present the main sets of I-sections used globally. Although South Africa's I-sections originated in Britain and Europe, they are added in separately, as they differ slightly.

4.7.1.1 European I-sections

There are six different sets of I-sections available in Europe. The different sets of European I-sections are as follows (ArcelorMittal, 2016):

- Parallel flange I-sections, known as European I-beams (IPE).

- Taper flange I-sections, known as European standard beams (IPN).
- European wide flange beams (HE).
- European extra wide flange beams (HL).
- Wide flange columns (HD).
- Wide flange bearing piles (HP).

4.7.1.2 British I-sections

Great Britain has separate sets of I-sections from the European I-sections. There are four different sets of British I-sections and they are as follows (ArcelorMittal, 2016):

- British universal beams (UB).
- Taper flange I-sections, known as British joists with taper flange (J).
- British universal columns (UC).
- British universal bearing piles with wide flanges (UBP).

4.7.1.3 American I-sections

The Americans classify their I-sections differently to most other countries. They do not classify their columns and beams separately, but combine them in one set, known as the American wide flange beams set (W). There are three different sets of American I-sections, which include all of their I-sections (ArcelorMittal, 2016). These sets are as follows (ArcelorMittal, 2016):

- American wide flange beams (W).
- American standard beams (S).
- American wide flange bearing piles (HP).

4.7.1.4 Russian I-sections

The Russians do not have a huge rage of hot-rolled I-sections. All of their I-sections are presented in one set, on ArcelorMittal's website (ArcelorMittal, 2016). The Russian set of I-sections is named the Russian hot rolled beams (HG) (ArcelorMittal, 2016).

4.7.1.5 Japanese I-sections

Japanese I-sections are grouped into one set, named the Japanese H sections (H). The dimensions of these I-sections can be obtained from ArcelorMittal's website (ArcelorMittal, 2016).

4.7.1.6 South African I-sections

As mentioned above, all of the available hot-rolled I-sections in South Africa originated in Great Britain and Europe. They are however slightly different from the European and British I-sections, probably due to the slightly different manufacturing processes. The dimensions and properties of the South African hot-rolled I-sections can be found in Appendix A and are divided into the following sets (SAISC, 2013):

- IPE sections
- Universal beams (UB)
- Universal columns (UC)
- Taper flange I-sections (J)

4.7.2 Methodology used to create Initial Set 1

The methodology used to create welded I-sections to approximate the set of hot-rolled I-sections available globally, involved three steps. These steps correspond to four dimensions of welded I-sections, namely:

- web thickness (t_w)
- flange thickness (t_f)
- web depth (h_w^*)
- flange width (b)

In Step 1 the web and flange thicknesses of each welded I-section were determined by rounding up the web and flange thickness of the corresponding hot-rolled I-section to the nearest available plate thickness (refer to Section 4.3.1). Any plate thickness less than 4 mm was rounded up to 4.5 mm, as plates smaller than 4 mm are not weldable, as mentioned in Section 4.3.3.

In Step 2 the web depth and flange widths of each welded I-section were determined by choosing a suitable plate size according to:

- the web depth and flange width of the corresponding hot-rolled I-section
- economical plate type (flat bars are more economical than other plate types)
- plate nesting (according to Section 4.3.2)

Flat bars were always the first choice of all the plate types, if the correct flat bar size was available, as no plate cutting is required when using flat bars, as mentioned in Section 4.3.1. Flat bars are therefore normally more economical than other plate types.

After Steps 1 and 2, 901 welded I-sections were created. Amongst these I-sections were some duplicates due to similar dimensions in different countries. In Step 3, these duplicates were removed from the set of welded I-sections, reducing the set to 692 sections.

The practical considerations mentioned in Section 4.3, which were not accounted for in the three steps above, were accounted for in the practicality tests of the optimisation methodology presented in Chapter 5.

After applying the practicality tests, the set of 692 I-sections were further reduced to 608 I-sections.

4.8 Initial Sets 2, 3 and 4

The I-sections in Initial Sets 2, 3 and 4 were created with the use of all the practical considerations mentioned in Section 4.3, excluding the plate nesting considerations mentioned in Section 4.3.2. During the process of creating Initial Set 1, it was found that the different web and flange plate sizes do not yield a large difference in material waste for most of the welded I-sections.

4.8.1 Methodology used to create Initial Set 2, 3 and 4

The I-sections of Initial Sets 2, 3 and 4 were created in four steps. Each step corresponds to a dimension of a welded I-section.

4.8.1.1 Step 1 - Web depth

The web depth of the I-sections was defined firstly, as it is not linked or dependent on any other dimension of the welded I-sections.

The web depths of the welded I-sections ranged from the minimum web depth ($(h_w)_{min}$) to the maximum web depth ($(h_w)_{max}$), in increments of either 10 or 50 mm. Refer to Section 4.4.3 for web depth ranges.

The 50 mm increments correspond closely to the incremental differences in web depth between the I-sections used in South Africa.

The initial set of I-sections created with a 10 mm web increment was used to test the sensitivity of the optimal sets of welded I-sections to the increment size.

4.8.1.2 Step 2 - Flange thickness

The flange thickness was defined next, as it is also not dependent on any other I-section dimension. The flange thicknesses ranged from 4.5 mm to 150 mm, with 36 different plate thicknesses to choose from. The practical considerations behind this range of thicknesses can be found in Section 4.4.1.

4.8.1.3 Step 3 - Web thickness

The web thickness was defined next and it is dependent on the flange thickness.

The same range of plate thicknesses were used for the web as for the flanges, as long as the following minimum and maximum web thickness requirements were met:

- minimum web thickness - the web thickness should not be less than 1/6 of the flange thickness or 5 mm
- maximum web thickness - the web thickness should not be more than the flange thickness

More information about why these requirements (limitations) were used, can be found in Section 4.4.1.

4.8.1.4 Step 4 - Flange width

The flange width was defined last due to its dependence on the flange thickness and web depth.

The flange width had to be smaller than or equal to the width defined in Equation 4.5 and larger than 100 mm (refer to Section 4.4.2). The flange widths were defined in increments of 10 mm or 20 mm.

The 20 mm flange increment closely corresponds with the flange widths increments of the I-sections currently used in South Africa.

The initial set of I-sections created with a 10 mm flange increment was used to test the sensitivity of the flange increment sizes on the optimal sets of welded I-sections.

4.8.2 Created sets of I-sections

As mentioned above, Initial Sets 2, 3 and 4 were created with different section increments and to different design parameters. Initial Sets 2 and 3 were created for beam conditions and Initial Set 4 was created for girder conditions.

Two initial sets were created for beam conditions and one set for girder conditions, as the two sets for beam conditions were sufficient to test the sensitivity of the optimal set of welded I-sections to the different section increments.

Initial Set 2 was created with the use of 10 mm web depth and flange width increments. This set consisted of 2,394,896 sections.

Initial Sets 3 and 4 were created with the use of a 50 mm web depth increment and a 20 mm flange width increment. Initial Set 3 consisted of 255,663 sections and Initial Set 4 of 872,735 sections.

4.9 Conclusion

The methodology presented in this chapter, in combination with the practicality tests presented in Chapter 5, provided the initial set of practical welded I-sections. These initial sets were used in the optimisation methodology, as presented in Chapter 5, to obtain the optimal sets of welded I-sections.

Chapter 5

Methodology to obtain an optimal set of welded I-sections

5.1 Introduction

The methodology presented in this chapter was used to obtain the optimal set of welded I-sections from the different initial sets discussed in Chapter 4.

The methodology followed seven steps, as explained in the following subsections. The complete methodology was carried out with the use of a computer program developed for this specific purpose by the researcher, as presented in Section 5.9.

5.2 Step 1 - Test practicality of I-sections

All the initial sets of welded I-sections were put through four tests to determine whether or not they were practical, as unpractical sections will never be used in practice.

Firstly, the flange width of the I-section was required to be larger than or equal to 100 mm, as it is difficult to drill holes in flanges narrower than 100 mm, making them unpractical (refer to Section 4.4.2).

Secondly, no Class 4 flanges were considered in this paper. According to SANS 10162-1 (SABS, 2011c), material is wasted when Class 4 flanges are used; making Class 4 flanges uneconomical (refer to Section 4.4.2).

Thirdly, the web and flange thickness of an I-section had to be thicker than 5 mm to ensure ease of welding (refer to Section 4.3.3).

Fourthly, the web slenderness was tested according to Section 14.3 of SANS 10162-1 (SABS, 2011c), which states that the slenderness ratio of a web may not exceed $83000/f_{yw}$ (refer to Section 4.4.4).

All sections that did not meet these limits were discarded.

5.3 Step 2 - Definition of design space and Workable Sets

In Step 2 I-sections were tested under different values of the design parameters, which define the design space, in order to obtain the Workable Set for a specific data point. The Workable Set contained all the sections which met resistance tests for each data point. The relevant design parameters are the lateral support conditions, span, effective length and loading.

The effective length, span and lateral support conditions combine to form the beam and girder conditions, while the loading can either be uniformly distributed loads (for beams) or point loads (for girders). The design parameters of beams and girders were distinguished not only in terms of their loading parameters, but also in terms of their design parameter ranges.

Two different design parameter ranges were defined for beams (Design Space A and B) and one for girders (Design Space C). The differences between the three design parameter ranges are defined in the following subsections.

5.3.1 Define BEAM conditions and loading

Beam conditions depend on two properties, beam span and lateral support, which in combination determine the effective length. Two lateral support conditions were considered: laterally supported and laterally unsupported beams.

The effective lengths were equal to either the beam span (for laterally unsupported conditions) or zero (for laterally supported conditions).

The beam spans ranged from 2 m to 10 m for Design Space A and from 0.5 m to 10 m for Design Space B, at intervals of 0.5 m.

Only uniformly distributed loads were applied to the I-sections in this part of the research project, as this is the most common loading for beams (refer to Section 1.4). The factored uniformly distributed loads ranged from 1 kN/m to 40 kN/m for Design Space A and from 1 kN/m to 45 kN/m for Design Space B, at intervals of 0.5 kN/m. The ranges of distributed loads and beam spans were determined as described in Chapter 7.

The weight of each beam was added separately to the specified factored uniformly distributed load.

The different beam conditions in combination with the factored uniformly distributed loads yielded 2686 different data points for Design Space A and 3560 different data points for Design Space B, each defined by a unique combination of span, load and lateral support conditions.

5.3.2 Define GIRDER conditions and loading

Girder conditions depend on three properties: girder span, effective length and lateral support. Both lateral support conditions were considered for girders, namely laterally supported and laterally unsupported.

The effective length of a girder is zero when it is laterally supported and equal to the specified effective length when is is laterally unsupported.

For Design Space C, the girder spans ranged from 1 m to 10 m and the effective lengths from 0.5 m to 6 m, at intervals of 0.5 m. However, the load spacing (beam spacing) had to fit into the effective length for the design parameter to be valid for Design Space C, as the effective length of a girder would normally (and economically) be multiples of the load spacing (beam spacing). For this reason an effective length of 6 m will for example not be specified with a load spacing of 2.5 m.

The load spacing ranged from 0.5 m to 4.5 m for Design Space C, at intervals of 0.5 m. The load spacing had to fit into the girder span for the design parameter to be valid for Design Space C, as that is how an engineer would normally specify his load spacing. For example, a beam spacing of 2.5 m will not normally be specified on a girder span of 6 m, because it would be unsymmetrical and take longer to design.

Only point loads were applied to the I-sections in this part of the research project, as it is inherent to the girder definition (refer to Section 1.4). The factored point loads ranged from 10 kN to 350 kN for Design Space C, at 10 kN intervals. The ranges of point loads, load spacing, girder spans and effective lengths were determined as described in Chapter 7.

The weight of each girder was also added separately to the factored point loads, as a factored uniformly distributed load.

The different girder conditions in combination with the factored point loads yielded 18060 different data points for Design Space C, each defined by a unique combination of span, effective length, load, load spacing and lateral support conditions.

5.3.3 Beam and Girder resistance tests to define Workable Set

For each data point, the moment and shear resistance of each I-section were calculated according to SANS 10162-1 (SABS, 2011c) and S16-14 (CSA Group, 2014). It was then compared with the ultimate moment and shear caused by the (ultimate limit state) distributed loads or point loads associated with that data point. The deflection was also checked not to exceed a value of $span/300$, based on a serviceability load equal to the ultimate load divided by 1.4.

The bearing resistance of the I-sections was tested according to SANS 10162-1 (SABS, 2011c) when girder design parameters were applied.

The equations used to calculate the applied loads, section capacity and stiffness can be obtained from Chapter 6.

If at any data point the capacities of a section exceeded the calculated moments and shear forces, and its deflection was within limits, the section was declared a member of the Workable Set corresponding to that data point.

5.4 Step 3 - Rating of sections according to weight

For each of the data points, the members of the Workable Set obtained in Step 2 were first sorted according to section weight, from lowest to highest weight.

After the sections were sorted, the I-section with the least weight at a given data point was given a rating of 100. Each other I-section forming part of the Workable Set at that data point obtained a rating calculated according to Equation 5.1.

$$R_{ij} = 100 - \frac{W_i - W_{Lj}}{W_{Lj}} \times 100 \tag{5.1}$$

where R_{ij} is the rating of section i at data point j, W_i is the weight of section i in kg/m and W_{Lj} is the weight of lightest section at data point j.

The sections with a rating less than the specified minimum rating (R_{min}) were then discarded from the Workable Sets to form the Rated Sets in Step 3, as shown below in Equation 5.2 and 5.3.

$$(i) \;\; If \qquad R_{ij} < R_{min} \quad \rightarrow \quad R_{ij} = 0 \tag{5.2}$$

$$(ii) \;\; If \qquad R_{ij} \geq R_{min} \quad \rightarrow \quad R_{ij} = R_{ij} \tag{5.3}$$

The minimum rating was varied between values of 75 and 100, at increments of 5, in order to test the sensitivity of the optimal set to the specified minimum ratings.

A minimum rating of 75 means that a section's weight had to be less than 1.25 times that of the lightest section at a given data point, otherwise its rating at that specific data point would be zero. For a member i not forming part of the Workable Set at data point j the same rule applied: $R_{ij} = 0$.

5.5 Step 4 - Applying weighting factors at each position in design space

Step 4 of the methodology took into account the fact that certain values of the design parameters occur more frequently in practice than others do. A given section will clearly be more valuable if it is an economical choice over a range of commonly occurring loads and spans, than when it is only a good choice for a combination of spans or loads that occurs rarely in practice.

In Step 4 the ratings calculated in Step 3 were factored according to the frequency of occurrence of the design parameters associated with each of the data points. These ratings were factored with weighting factors corresponding to each of the possible parameters, for example the span, loading, etc. These weighting factors indicate how frequently a loading parameter will occur in practice. In other words, how popular a design parameter will be in practice.

A weighting factor of 1.0 was specified for the design parameters that occur frequently in practice, while a weighting factor of 0.0 was specified for the design parameters that hardly ever occur in practice. A weighting factor of 0.5 thus means that the load parameter occurs half as often as the load parameter with a weighting factor of 1.0.

The factored rating (R^*_{ij}) of each I-section for each of the data points was calculated based on Equation 5.4 below.

$$R^*_{ij} = R_{ij} \cdot f_{support.j} \cdot f_{span.j} \cdot f_{effective.j} \cdot f_{load.j} \cdot f_{spacing.j} \tag{5.4}$$

where R_{ij} is the rating of section i at data point j according to Equation 5.1, $f_{support.j}$ is the lateral support weighting factor, $f_{span.j}$ is the span weighting factor, $f_{effective.j}$ is the effective length weighting factor, $f_{load.j}$ is the load weighting factor and $f_{spacing.j}$ is the load spacing weighting factor, all for data point j.

5.5.1 Weighting factors for beam parameters

Design Space A and B did not have effective length weighting factors ($f_{effective}$), because the effective length of a beam is directly linked to its lateral support conditions, as mentioned in Section 5.3.1. These sets also did not have load spacing weighting factors ($f_{spacing}$), as these weighting factors are only used in combination with weighting factors specified for point loads, which is not the case for beams. Only uniformly distributed loads were considered for beams. Equation 5.4 could thus be reduced to the following equation for beam parameters:

$$R^*_{ij} = R_{ij} \cdot f_{support.j} \cdot f_{span.j} \cdot f_{load.j} \tag{5.5}$$

The weighting factors of Equation 5.5 for Design Space A and B are presented in Section 7.4 and calculated on the basis of the information presented in Chapter 7.

5.5.2 Weighting factors for girder parameters

The factored ratings of Design Space C were calculated with the use of Equation 5.4, as girders have all the specified parameters. The weighting factors of Design Space C can be obtained from Section 7.5 and were calculated on the basis of the information presented in Chapter 7.

5.6 Step 5 - Final ranking of I-sections to form Ranked Set

The final ranking (K_i^*) of I-sections was determined by adding up, for each section, its factored ratings for each of the data points in the design space:

$$K_i^* = \sum_{j=1}^{n} R_{ij}^* \tag{5.6}$$

where n is the number of data points.

The Ranked Set was obtained by compiling all the sections which had a final ranking higher than zero.

5.7 Step 6 - Removing similar I-sections from Ranked Set to find Initial Optimal Set

Amongst the I-sections in the Ranked Set were some sections with similar dimensions, constituting duplication. The occurrence of similar sections in the Ranked Set was reduced by comparing them with each other and removing the least popular I-sections.

Two different methods were initially considered to compare the sections to each other. The first method only focused on whether sections had very similar geometry and weight. The second method compared the I-sections by considering their degree of overlap in the design space.

Upon evaluation, the second method was found to provide more reliable and better results, as it did not only compare I-sections with similar geometry, but also sections with similar properties. The second method was therefore used to determine all of the initial optimal sets.

The first step in the second method was to identify all the overlapping data points of any two sections of the Ranked Set, i.e. all the data points at which both have a rating other than zero. When these points exceeded a predetermined percentage (the allowable overlap percentage) for one of the two sections, the one with the lower final ranking was removed from the set. The only exception made in this regard was when a section was the only one with a rating above zero at any of the data points in the design space, in which case it was retained in the set.

Figure 5.1 provides an example of a typical comparison between sections. Section A works for a large region of the design space and will have a higher final ranking. Data points of Section B overlaps 100 % with the data points of Section A. Section B will thus be discarded with the mentioned second method if the allowable overlap percentage is 100 %, because Section A has a higher final ranking than Section B. Sections C and D will not be discarded as their overlap percentages is lower than the allowable overlap percentage of 100 %. However, if an allowable overlap percentage of 50 % is defined and Section A has a higher final ranking than Section D, Section D will be discarded, as the data points of Section D overlaps more than 50 % with Section A.

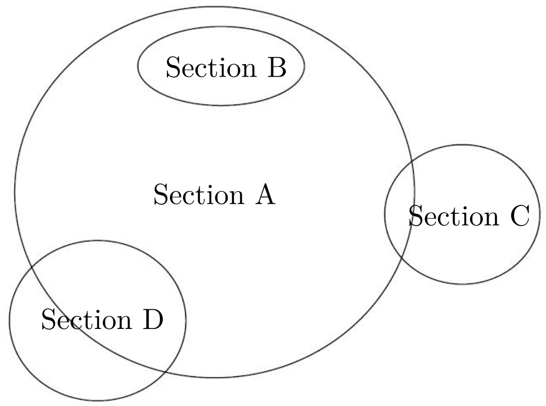


Figure 5.1: Example of overlapping data points

The reduction process in this optimisation methodology started with a allowable overlap percentage of 100 %, after which the percentage was reduced gradually in increments of 2.5 % or 5 % depending on the sensitivity of the optimal set of I-sections. The allowable overlap percentage was then gradually diminished in order to reduce the optimal set to about 70 to 80 sections. An allowable overlap percentage of less than 70 % was however never used, as the size of the set does not reduce significantly with a smaller allowable overlap percentage.

The sensitivity of the sets to the allowable overlap percentage was evaluated by assessing the section distribution over the design space. When all the popular design problems had at least one section that worked for it, the allowable overlap percentage increment was regarded as acceptable. When no section worked for one of the design problems, which were covered before, the allowable overlap percentage had to be adjusted.

A size limit of between 70 and 80 sections per set was decided upon for beam conditions, as that is approximately the same as the number of I-sections that have historically been available for beams in South Africa, as listed in the Red Book (SAISC, 2013).

A size limit of approximately 150 sections per set was decided upon for girder conditions, as there are about 150 sections that have historically been available for girders in South Africa, as listed in the Red Book (SAISC, 2013).

The Ranked Set from Step 5 was reduced to the Initial Optimal Set of welded I-sections in Step 6.

5.8 Step 7 - Obtain Final Optimal Set

The Initial Optimal Set typically contained more than one section for the data points in the popular region of the design space, but in practice only the lightest section for a specific design problem (data point) will ever be used. For this reason the final optimal set has to be the lightest possible set from the Initial Optimal Set of welded I-sections.

The Initial Optimal Set of sections were reduced to the lightest possible set by repeating Step 2 to 6, with a minimum rating of 100, using the Initial Optimal Set as the Initial Set. This yielded the final optimal set of welded I-sections.

5.9 Optimisation program

As mentioned in Section 5.1, the whole optimisation methodology was carried out with the use of a computer program developed for this purpose. This section provides a brief overview of this program, that was written by the researcher in the computer language Java.

5.9.1 Program outline

The program consisted of a main program which made use of some subroutines to carry out the optimisation methodology, as presented in this chapter. The flow of this program is shown by the flow diagram in Figure 5.2. The description of the coloured shapes is provided in Table 5.1. The thick arrows in Figure 5.2 represent the main flow of the program, the normal arrows the secondary flow paths and the dashed arrows represent the flow of information between the subroutines. The flow diagram is described in Section 5.9.2.

Table 5.1: Description of the coloured shapes in the flow diagram

Coloured shapes	**Representation of**
Red rectangle	the start and stop
Blue trapezium	the input and output
Orange rectangle	the processes
Green diamond	the decisions

5.9.2 Program flow description

The flow of the program is described in the following subsections by referring to the different steps in the optimisation methodology and the flow diagram in Figure 5.2.

5.9.2.1 Input

The program required three different inputs at the start of the program in order to obtain the optimal set of welded I-sections, namely the initial section sizes, the defined design space and the corresponding popularity weighting factors.

The initial section sizes (block (a) in Figure 5.2) corresponds to the minimum and maximum dimensions specified in Chapter 4, allowing the creation of the initial sets of welded I-sections.

This was only required for Initial Sets 2, 3 and 4. When an initial set was already defined, as with Initial Set 1 and Step 7 of the optimisation methodology (Section 5.8), the input of section sizes was bypassed and an initial set of sections used instead.

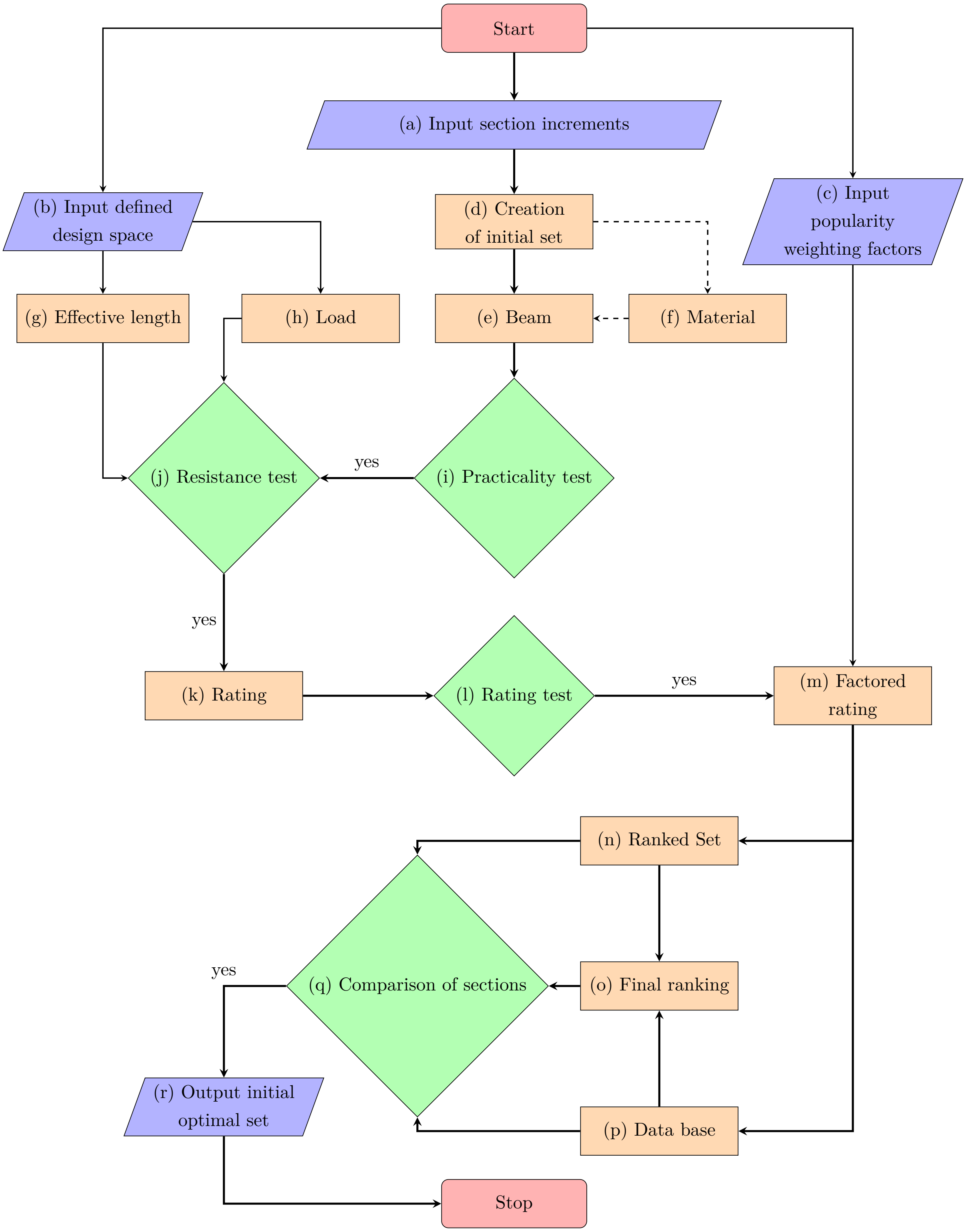


Figure 5.2: Flow diagram of optimisation program

The defined design space or design parameters (block (b) in Figure 5.2) were defined as per Step 2 of the optimisation methodology (refer to Section 5.3) and the corresponding popularity weighting factors (block (c) in Figure 5.2) in Chapter 7.

5.9.2.2 Creation of initial set of welded I-sections

A process was used to create the initial set of welded I-sections (block (d) in Figure 5.2) from the initial section sizes, as stipulated in Chapter 4. This processes defined the section dimensions and properties of the initial set, as well as the yield stress of the flanges and webs of the I-sections, as the yield stress is a function of the thickness of these dimensions.

When the initial set was already created with another processes, as is the case with Initial Set 1, its section dimensions and properties were imported, instead of calculating them.

The Material process (block (f) in Figure 5.2) took the yield stresses of each section from the initial set and stored it with the other constant material properties (refer to Section 4.6) in a material subroutine for each section. These material properties of each section could then be recalled from the material subroutine.

The Beam process (block (e) in Figure 5.2) took the dimensions and properties of the sections from the initial set and stored the values with the corresponding material subroutine in a beam subroutine for each section. The dimensions and properties (section and material) of each section could then be recalled from the beam subroutine.

5.9.2.3 Step 1 of the optimisation process

These beam subroutines were used in the Practicality test decision process (block (i) in Figure 5.2) to test the practicality of the initial I-sections, according to Step 1 of the optimisation methodology (refer to Section 5.2). The sections declared practical were then sent to the Resistance test decision process. The sections that did not meet the practicality requirements were discarded.

5.9.2.4 Step 2 of the optimisation process

The Effective length and Load process corresponds to the definition of the design parameters in Step 2 of the optimisation methodology. The Effective length process (block (g) in Figure 5.2) stored the beam or girder parameters (effective length and span) of each data point of the defined design space (block (b) in Figure 5.2) in a corresponding effective length subroutine and the Load process (block (h) in Figure 5.2) stored the load parameters (distributed or point loads) of each data point in a corresponding load subroutine. These parameters could then be recalled from these subroutines. The load subroutine of each data point also calculated the applied loads (applied moment, shear, etc.) corresponding to the data point.

These mentioned subroutines were then used with the beam subroutines of the practical I-sections to obtain the Workable Set (refer to Section 5.3) of each data point in the Resistance test decision process (block (j) in Figure 5.2). This was done by calculating the capacity of each practical I-section and discarding the sections which did not have sufficient capacity at that data point. The Workable Sets were then send to the Rating Process (block (k) in Figure 5.2).

The capacity of the sections was calculated for each data point in the Resistance test decision process with the use of a section capacity subroutine. This subroutine made use of the effective length subroutine of the Effective length process (block (g) in Figure 5.2) and the beam subroutine from the Practicality test (block (i) in Figure 5.2), to calculate the moment, shear and bearing resistance (if required) of the practical sections at each of the data points.

5.9.2.5 Step 3 of the optimisation process

The Rating process (block (k) in Figure 5.2) and Rating test (block (l) in Figure 5.2) corresponds to Step 3 of the optimisation process (refer to Section 5.4).

The Rating process calculated the rating of all the sections in the different Workable Sets, according to Section 5.4. The Rating test then gave the sections a rating of zero when its rating was smaller than the minimum rating, which effectively meant that the rating of the section was discarded when it was not larger or equal to the minimum rating.

5.9.2.6 Step 4 of the optimisation process

The Factored rating process (block (m) in Figure 5.2) calculates the factored rating of all the sections, according to Step 4 of the optimisation process (refer to Section 5.5). The ratings were obtained from the Rating test decision process (block (l) in Figure 5.2) and the popular weighting factors from the input weighting factors (block (c) in Figure 5.2). The process produced a list of sections which worked for each data point with its corresponding factored rating.

5.9.2.7 Step 5 of the optimisation process

The Ranked Set process (block (n) in Figure 5.2), the Final ranking process (block (o) in Figure 5.2) and the Data base process (block (p) in Figure 5.2) corresponds to Step 5 of the optimisation process.

The lists of sections obtained from the Factored rating process were compared to each other in the Ranked Set process to determine which sections work, over the entire design space. These sections were then stored in one set, the Ranked Set.

The lists of sections from the Factored rating process were also used in the Data base process to determine for which data points each section work over the design space. These data points with their factored rating were then stored in a data base corresponding to each I-section in the Ranked Set.

The final ranking of each section in the Ranked Set were then calculated in the Final ranking process by simply adding up, for each section, its factored ratings for each of the data points present in the corresponding data base. These final rankings were then sent to the Comparison of sections decision process (block (q) in Figure 5.2).

5.9.2.8 Step 6 of the optimisation process

The Comparison of sections decision process (block (q) in Figure 5.2) corresponds to Step 6 of the optimisation process. This process removed similar I-sections from the Ranked set in accordance to Section 5.7 and produced the Initial Optimal Set of I-sections (block (r) in Figure 5.2).

5.10 Conclusion

The methodology and corresponding computer program presented in this chapter produced a set of welded I-sections optimised according to minimum weight, that meets the capacity and deflection requirements of SANS 10162-1 (SAISC, 2013) over the design space and that is fabricatable and economically viable. The methodology is thus logical and consistent with industry demands for economy and practicality.

As the set covers the popular design parameters in the design space, it will not only be optimal, but also popular in practice.

Chapter 6

Applied loads and section capacities

6.1 Introduction

The methodology presented in Chapter 5 tested the capacity of the I-sections under various values of the design parameters with different applied loads. This chapter provides the equations used to obtain these applied loads, as well as the section capacities, deflection limits and stiffness of the I-sections.

6.2 Applied loads

Before any applied loads were calculated, the uniformly distributed loads and point loads corresponding to each of the data points, mentioned in Section 5.3, were calculated for the ultimate limit state (ULS) and the serviceability limit state (SLS) for each I-section.

6.2.1 Applied loads on beams

6.2.1.1 Factored uniformly distributed loads

The own weight of each beam was used in combination with the factored distributed loads (refer to Section 5.3.1) to calculate the ULS and SLS distributed loads, as demonstrated by the following equations:

$$w_{ULS} = w + 1.2 \cdot w_{beam} \tag{6.1}$$

$$w_{SLS} = \frac{w}{1.4} + 1.1 \cdot w_{beam} \tag{6.2}$$

where w_{ULS} is the uniformly distributed load at the ultimate limit state $[kN/m]$, w_{SLS} is the uniformly distributed load at the serviceability limit state $[kN/m]$, w is the factored uniformly distributed load (discussed in Section 5.3.1) and w_{beam} is the own weight of the beam $[kN/m]$.

The load factors of 1.2 and 1.1 were obtained from SANS 10160-2 (SABS, 2011b). The factored distributed loads were divided by 1.4 for serviceability limit state, as they were originally factored for the ultimate limit state (refer to Section 7.4). The 1.4 factor is the average of the 1.2 permanent load factor and the 1.6 imposed load factor, as per SANS 10160-2 (SABS, 2011b).

6.2.1.2 Applied moments

The following equations were used to calculate the applied moments in the I-beams, as this research project only focused on simply supported beams. These applied moments were used to calculate the ω_2 values as per Section 6.3.1 and to determine if the I-sections had sufficient resistance.

$$M_{max} = M_b = \frac{w_{ULS} \cdot L^2}{8} \tag{6.3}$$

$$M_{min} = 0.0 \tag{6.4}$$

$$M_a = M_c = \frac{3}{32} \cdot w_{ULS} \cdot L^2 \tag{6.5}$$

where M_{max} is the maximum factored bending moment over the beam span $[kN.m]$, M_{min} is the minimum factored bending moment over the beam span $[kN.m]$, M_a is the factored bending moment at one-quarter point of the beam span $[kN.m]$, M_b is the factored bending moment at midpoint of the beam span $[kN.m]$, M_c is the factored bending moment at three-quarter point of the beam span $[kN.m]$ and L is the beam span $[m]$.

6.2.1.3 Applied shear

The following equation was used to calculate the maximum applied shear force (V_{max}) in the simply supported beams:

$$V_{max} = \frac{w_{ULS} \cdot L}{2} \qquad [kN] \tag{6.6}$$

6.2.2 Applied loads on girders

The moment and shear forces varies in each girder according to the different applied point loads and load spacing. For this reason more equations were used to calculate the applied loads on girders than what were used for beams.

The girder load parameters were broken up into four groups to simplify the calculations without compromising the accuracy of the calculations too much.

The design parameters fell into **Group 1** when the load spacing was less than a quarter of the girder span and into **Groups 2, 3 and 4** when the load spacing was larger.

The girder spans of Group 2 were equal to two times the load spacing, three times for Group 3 and four times for Group 4.

6.2.2.1 Factored girder loads

The applied loads of Group 1 were determined by converting the applied point loads to an equivalent distributed load. The applied loads of Group 1 were then determined using the same equations as for beams (refer to Section 6.2.1), with the factored uniformly distributed load (w) calculated with Equation 6.7.

$$w = \frac{1.2 \cdot \left(\frac{L}{spacing} - 1.0\right) \cdot P}{L} \tag{6.7}$$

where L is the girder span $[m]$, P is the factored point load (mentioned in Section 5.3) $[kN]$ and $spacing$ is the load spacing $[m]$.

For Groups 2, 3 and 4, the applied loads were first calculated separately to determine applied point loads and the own weight of the girders. They were then combined through superposition. The ULS and SLS loads of Groups 2, 3 and 4 were calculated as follows:

$$w_{girderULS} = 1.2 \cdot w_{girder} \tag{6.8}$$

$$w_{girderSLS} = 1.1 \cdot w_{girder} \tag{6.9}$$

$$P_{SLS} = \frac{P}{1.4} \tag{6.10}$$

where $w_{girderULS}$ is the uniformly distributed load of the girder at ultimate limit state $[kN/m]$, $w_{girderSLS}$ is the uniformly distributed load of the girder at serviceability limit state $[kN/m]$, w_{girder} is the own weight of the girder $[kN/m]$ and P_{SLS} is the point load at serviceability limit state $[kN]$.

6.2.2.2 Applied moments

The following equations were used to determine the applied moments on the girders from Groups 2, 3 and 4, as only simply supported girders were considered in this research project. These applied moments were also used to calculate the ω_2 values in Section 6.3.1 when the effective length of the girders corresponded to the load spacing. When the effective length did not correspond to the load spacing, the required moments were calculated over the effective lengths to determine the ω_2 values.

$$M_{min} = 0.0 \tag{6.11}$$

Group 2:

$$M_{max} = M_b = \frac{w_{girderULS} \cdot L^2}{8} + \frac{P \cdot L}{4} \tag{6.12}$$

$$M_a = M_c = \frac{3}{32} \cdot w_{girderULS} \cdot L^2 + \frac{P \cdot L}{8} \tag{6.13}$$

Group 3:

$$M_{max} = M_b = \frac{w_{girderULS} \cdot L^2}{8} + \frac{P \cdot L}{3} \tag{6.14}$$

$$M_a = M_c = \frac{3}{32} \cdot w_{girderULS} \cdot L^2 + \frac{P \cdot L}{4} \tag{6.15}$$

Group 4:

$$M_{max} = M_b = \frac{w_{girderULS} \cdot L^2}{8} + \frac{P \cdot L}{2} \tag{6.16}$$

$$M_a = M_c = \frac{3}{32} \cdot w_{girderULS} \cdot L^2 + \frac{3 \cdot P \cdot L}{8} \tag{6.17}$$

where M_{max} is the maximum factored bending moment over the girder span $[kN.m]$, M_{min} is the minimum factored bending moment over the girder span $[kN.m]$, M_a is the factored bending moment at one-quarter point of the girder span $[kN.m]$, M_b is the factored bending moment at midpoint point of the girder span $[kN.m]$, M_c is the factored bending moment at three-quarter point of the girder span $[kN.m]$ and L is the girder span $[m]$.

6.2.2.3 Applied shear

The following equations were used to calculate the maximum applied shear force (V_{max}), in kN, on the simply supported girders from Groups 2 to 4.

Group 2:

$$V_{max} = \frac{w_{girderULS} \cdot L}{2} + \frac{P}{2} \tag{6.18}$$

Group 3:

$$V_{max} = \frac{w_{girderULS} \cdot L}{2} + P \tag{6.19}$$

Group 4:

$$V_{max} = \frac{w_{girderULS} \cdot L}{2} + \frac{3 \cdot P}{2} \tag{6.20}$$

6.3 Section capacity

The moment, shear and weld capacity of welded I-sections had to be tested in Step 2 of the optimisation methodology (refer to Section 5.3). When the I-sections were optimised under girder design parameters, the bearing resistance had to be tested too.

6.3.1 Moment resistance

The I-sections firstly needed to be classified according to SANS 10162-1 (SABS, 2011c), after which the moment resistance of the sections could be calculated according to SANS 10162-1 (SABS, 2011c).

6.3.1.1 Section classification

The width-to-thickness ratios of flanges and webs were used to classify the I-sections, according to Table 4 of SANS 10162-1 (SABS, 2011c). These ratios were calculated as follows, as explained in Section 11.3 of SANS 10162-1 (SABS, 2011c):

$$\left(\frac{b}{t}\right)_{flange} = \frac{0.5 \cdot b}{t_f} \tag{6.21}$$

$$\left(\frac{b}{t}\right)_{web} = \frac{h_w^*}{t_w} \tag{6.22}$$

where $(b/t)_{flange}$ is the flange with-to-thickness ratio, $(b/t)_{web}$ is the web with-to-thickness ratio, b is the flange width, h_w^* is the web depth, t_f is the flange thickness and t_w is the web thickness.

With the use of the with-to-thickness ratios, the I-sections were classified according to Table 4 of SANS 10162-1 (SABS, 2011c). The section classes are demonstrated in Table 6.1.

Table 6.1: I-section classification for flexural compression

Section Class	Flange of I-section	Web of I-section
Class 1	$\frac{0.5 \cdot b}{t_f} \leq \frac{145}{\sqrt{f_{yf}}}$	$\frac{h_w^*}{t_w} \leq \frac{1100}{\sqrt{f_{yw}}} \cdot \left(1 - 0.39 \cdot \frac{C_u}{\phi \cdot C_y}\right)$
Class 2	$\frac{0.5 \cdot b}{t_f} \leq \frac{170}{\sqrt{f_{yf}}}$	$\frac{h_w^*}{t_w} \leq \frac{1700}{\sqrt{f_{yw}}} \cdot \left(1 - 0.61 \cdot \frac{C_u}{\phi \cdot C_y}\right)$
Class 3	$\frac{0.5 \cdot b}{t_f} \leq \frac{200}{\sqrt{f_{yf}}}$	$\frac{h_w^*}{t_w} \leq \frac{1900}{\sqrt{f_{yw}}} \cdot \left(1 - 0.65 \cdot \frac{C_u}{\phi \cdot C_y}\right)$

where f_{yf} is the specified minimum yield stress of the flange $[MPa]$, f_{yw} is the specified minimum yield stress of the web $[MPa]$, C_u is the ultimate compression force in the member (assumed to be zero), C_y is the axial compression force in the member at the yield stress $[C_y = A \cdot f_{yw}]$ and A is the area of the section.

6.3.1.2 Moment resistance of laterally supported members

Equations 6.23 to 6.25 were used to calculate the factored moment resistance (M_r) of a welded I-sections when its compression flange was effectively continuously laterally supported in accordance with Section 13.5 of SANS 10162-1 (SABS, 2011c).

The Z_{pl} and Z_e about the x-axis was used to calculate the moment resistance of the I-sections, as only beam and girder design parameters were considered in this research project. The f_y value was also conservatively used as the f_{yf} value, as the moment is mainly carried by the flanges. The f_{yf} value will always be equal to or smaller than the f_{yw} value (refer to Sections 4.4 and 4.6).

a) for class 1 and class 2 sections:

$$M_r = \phi \cdot Z_{pl} \cdot f_y = \phi \cdot M_p \tag{6.23}$$

b) for class 3 sections:

$$M_r = \phi \cdot Z_e \cdot f_y = \phi \cdot M_y \tag{6.24}$$

c) for class 4 sections:

Only sections with class 4 webs were considered in this research project, excluding sections with class 4 flanges, as mentioned in Section 5.2.

According to SANS 10162-1 (SABS, 2011c), beams or girders with flanges that meet the requirements of class 3, but whose webs do not meet the requirements of class 3, should be designed in accordance with clause 14 of SANS 10162-1 (SABS, 2011c).

When the web slenderness ratio (h_w^*/t_w) of an I-section exceeds $1900/\sqrt{M_u \cdot I \cdot \phi \cdot Z_e}$ and the flange meets the requirements for class 3, the moment resistance must be calculated according to Equation 6.25 (SABS, 2011c).

$$M_r' = M_r \cdot \left[1 - 0.0005 \cdot \frac{A_w}{A_f} \cdot \left(\frac{h_w^*}{t_w} - \frac{1900}{\sqrt{M_u \cdot I \cdot \phi \cdot Z_e}}\right)\right] \tag{6.25}$$

where M_r is the factored moment resistance according to Equation 6.24.

6.3.1.3 Moment resistance of laterally unsupported members

When the I-sections were under laterally unsupported conditions (conditions where no continuous lateral support is provided) their moment resistances (M_r) were calculated according to the procedure and equations presented in this section.

S16-14 (CSA Group, 2014) was used to obtain the moment resistance of the welded I-sections under beam design parameters, because the S16-14 (CSA Group, 2014) is regarded to be more accurate than the South African steel code for beams under uniformly distributed loads.

The only difference between the two steel design codes is how they calculate the ω_2 coefficient, which is used in the process of calculating the factored moment resistance (M_r).

Equation 6.26 was used to calculate ω_2 for I-sections under uniformly distributed loads (CSA Group, 2014).

$$\omega_2 = \frac{4 \cdot M_{max}}{\sqrt{M_{max}^2 + 4 \cdot M_a^2 + 7 \cdot M_b^2 + 4 \cdot M_c^2}} \leq 2.5 \tag{6.26}$$

where M_{max} is the maximum factored bending moment in the unbraced length, M_a is the factored bending moment at one-quarter point of the unbraced length, M_b is the factored bending moment at midpoint of the unbraced length and M_a is the factored bending moment at three-quarter point of the unbraced length. Refer to Section 6.2 for moment values.

Equation 6.27 was used to calculate the ω_2 coefficient when the bending moment distribution within the unbraced length was effectively linear and the maximum bending moment (M_{max}) was not at any point within the unbraced length, but at the ends of the unbraced length, as is the case with some girder design parameters (SABS, 2011c).

$$\omega_2 = 1.75 + 1.05 \cdot \kappa + 0.3 \cdot \kappa^2 \leq 2.5 \tag{6.27}$$

where κ is the ratio of the smaller factored moment (M_{min}) to the larger factored moment (M_{max}) at the opposite ends of the unbraced length. The κ value is positive for double curvature and negative for single curvature.

The critical elastic moment (M_{cr}) was calculated according to Equation 6.28, which was obtained from SANS 10162-1 (SABS, 2011c).

$$M_{cr} = \frac{\omega_2 \cdot \pi}{K \cdot L} \cdot \sqrt{E \cdot I_y \cdot G \cdot J + \left(\frac{\pi \cdot E}{K \cdot L}\right)^2 \cdot I_y \cdot C_w} \tag{6.28}$$

where KL is the effective length of the beam segment and KL is assumed to be 1.2 times the beam span (L).

The factored moment resistance (M_r) of an I-section was calculated according to Equation 6.29 to 6.32, depending on the class of the I-section (SABS, 2011c).

a) Class 1 and 2 I-sections:

i) when $M_{cr} > 0.67 \cdot M_p$

$$M_r = 1.15 \cdot \phi \cdot M_p \left(1 - \frac{0.28 \cdot M_p}{M_{cr}}\right) \leq \phi \cdot M_p \tag{6.29}$$

ii) when $M_{cr} \leq 0.67 \cdot M_p$

$$M_r = \phi \cdot M_{cr} \tag{6.30}$$

b) Class 3 and 4 I-sections:

i) when $M_{cr} > 0.67 \cdot M_y$

$$M_r = 1.15 \cdot \phi \cdot M_y \left(1 - \frac{0.28 \cdot M_y}{M_{cr}}\right) \leq \phi \cdot M_y \tag{6.31}$$

ii) when $M_{cr} \leq 0.67 \cdot M_y$

$$M_r = \phi \cdot M_{cr} \tag{6.32}$$

iii) Use Equation 6.25 for class 4 sections, with M_r from Equation 6.31 or 6.32.

6.3.2 Shear resistance

The shear resistances (V_r) of the I-sections were calculated in accordance with Section 13.4 of SANS 10162-1 (SABS, 2011c), with Equation 6.33.

$$V_r = \phi \cdot A_v \cdot f_s \tag{6.33}$$

where A_v is the shear area ($t_w \cdot h_w^*$ for welded I-sections and $t_w \cdot h$ for hot-rolled I-sections) and f_s is the ultimate shear stress (calculated according to Equation 6.34 to 6.37).

The ultimate shear stress (f_s) is dependent on the web depth to thickness ratio (h_w^*/t_w) and can be calculated as follows:

a) When $\frac{h_w^*}{t_w} \leq 440 \cdot \sqrt{\frac{5.34}{f_{yw}}}$

$$f_s = 0.66 \cdot f_{yw} \tag{6.34}$$

b) When $440 \cdot \sqrt{\frac{5.34}{f_{yw}}} < \frac{h_w^*}{t_w} \leq 620 \cdot \sqrt{\frac{5.34}{f_{yw}}}$

$$f_s = f_{cri} \tag{6.35}$$

where:

$$f_{cri} = 290 \cdot \frac{\sqrt{5.34 \cdot f_{yw}}}{(h_w^*/t_w)} \tag{6.36}$$

c) When $620 \cdot \sqrt{\frac{5.34}{f_{yw}}} < \frac{h_w^*}{t_w}$

$$f_s = f_{cre} \tag{6.37}$$

where:

$$f_{cre} = \frac{961200}{(h_w^*/t_w)^2} \tag{6.38}$$

6.3.3 Bearing resistance

Bearing resistance (B_r) was only checked for girders, as it was assumed that beams could sometimes be carried on top of girders, rather than framing into their webs. It was calculated in accordance to Section 14.3.2 of SANS 10162-1 (SABS, 2011c), as follows:

a) for interior loads (Loads applied at a distance greater than the member depth from the member edge or end), B_r is the smaller of Equation 6.39 and 6.40:

$$B_r = \phi_{bi} \cdot t_w \cdot f_y \cdot (N + 10 \cdot t_f) \tag{6.39}$$

$$B_r = 1.45 \cdot \phi_{bi} \cdot {t_w}^2 \sqrt{f_y \cdot E} \tag{6.40}$$

b) for end reactions, B_r is the smaller of Equation 6.41 and 6.42:

$$B_r = \phi_{be} \cdot t_w \cdot f_y \cdot (N + 4 \cdot t_f) \tag{6.41}$$

$$B_r = 0.6 \cdot \phi_{be} \cdot {t_w}^2 \sqrt{f_y \cdot E} \tag{6.42}$$

where N is length of bearing, ϕ_{bi} is 0.80 and ϕ_{be} is 0.75.

Different N values were assumed for different factored point load ranges (P). These N values are provided in Table 6.2 and were assumed after a study was conducted on the moment resistance of the popular I-beams (presented in Section 7.3.1.1), as shown in Table 6.3.

The moment resistance about the x-axis (M_{rx}) and maximum spans (L) values of the popular I-beams shown in Table 6.3 were obtained from Table 5.5 of the Red Book (SAISC, 2013) and the moment resistance values were for laterally supported I-sections.

The factored point loads (P) correspond to the beam reactions supported by girders. These beam reactions for the popular I-beams are provided in Table 6.3 and were calculated with Equation 6.43. This equation was derived from the maximum applied moment equation for simply supported beams (refer to Equation 6.3).

$$Beam\ reaction = \frac{4 \cdot M_{rx}}{L} \tag{6.43}$$

where M_{rx} moment resistance around the x-axis (refer to Table 6.3) and L is the maximum span (refer to Table 6.3).

The lower P value limit (70 kN) was assumed as the rough average between the beam reactions of the IPE 200 and 305 x 102 x 25 I-section, which has a flange width of approximately 100 mm (refer to Table 6.3). The middle P value limits were assumed to be between 70 kN and 180 kN, as the capacity of the sections with medium size flanges (approximately 140 mm) can vary a great deal with different spans. The N value of 175 mm was assumed for the higher P value limit, as the larger popular I-sections have flange widths close to 175 mm, as with the 406 x 178 x 54 I-section (refer to Table 6.3).

Table 6.2: N values used in optimisation methodology

P values (kN)	**N values (mm)**
$P \leq 70$	100
$70 < P \leq 180$	140
$180 < P$	175

Table 6.3: Beam reaction forces of popular I-beams

Popular I-beams	**Flange width (mm)**	M_{rx} **(kN.m)**	**Maximum span (m)**	**Reactions (kN)**
IPE_{AA} 160	82	30	4	30.00
IPE 160	82	39.1	4	39.10
IPE_{AA} 200	100	55.4	4	55.40
IPE 200	100	69.6	5	55.68
203 x 133 x 25	133.2	81.6	6	54.40
254 x 146 x 31	146.1	124	7	70.86
305 x 102 x 25	101.8	106	5	84.80
305 x 165 x 40	165.1	197	9	87.56
356 x 171 x 45	171	243	10	97.20
406 x 140 x 39	141.8	226	8	113.00
406 x 178 x 54	177.6	331	11	120.36
457 x 191 x 67	189.9	463	11	168.36

6.3.4 Weld resistance

The weld resistance was deemed sufficient in this research project, as the weld size varies with different applied loads and is unlikely to influence the final optimal set. A weld size could potentially, but conservatively, be specified according to the weld rule of thumb, which ensures that the weld never yields before the web (refer to Section 3.13).

6.4 Displacement limit and resistance

The displacement resistance of each I-section was calculated with the equations presented in the following subsections and tested to a displacement limit of *span*/300, which corresponds to the displacement limit of simple span members supporting floors in industrial-type buildings, as per Annex D of SANS 10162-1 (SABS, 2011c).

6.4.1 Displacement of beams

The displacement equation for simply supported beams was obtained from Table 5.19 of the Red Book (SAISC, 2013) and was as follows:

$$\Delta_{max} = \frac{5}{384} \cdot \frac{w_{SLS} \cdot L^4}{E \cdot I} \tag{6.44}$$

where Δ_{max} is the maximum displacement, L the beam span, E the elastic modulus of steel and I the moment of inertia about the x-axis.

6.4.2 Displacement of girders

The displacement equations for simply supported girders were obtained by using Table 5.19 of the Red Book (SAISC, 2013). For the different girder groups, as defined in Section 6.2.2, the following equations were applied:

Group 1:

The displacement of the Group 1 girders was calculated with the same equation used for beams (Refer to Section 6.4.1).

Group 2:

$$\Delta_{max} = \frac{5}{384} \cdot \frac{w_{girderSLS} \cdot L^4}{E \cdot I} + \frac{P_{SLS} \cdot L^3}{48 \cdot E \cdot I} \tag{6.45}$$

Group 3:

$$\Delta_{max} = \frac{5}{384} \cdot \frac{w_{girderSLS} \cdot L^4}{E \cdot I} + \frac{23 \cdot P_{SLS} \cdot L^3}{648 \cdot E \cdot I} \tag{6.46}$$

Group 4:

$$\Delta_{max} = \frac{5}{384} \cdot \frac{w_{girderSLS} \cdot L^4}{E \cdot I} + \frac{19 \cdot P_{SLS} \cdot L^3}{384 \cdot E \cdot I} \tag{6.47}$$

Chapter 7

Field work and popularity weighting factors

7.1 Introduction

The purpose of this chapter is to define the popularity weighting factors (refer to Section 5.5) for each of the parameters that determines the configuration and design parameters of a beam: its span, loading, end constraints and conditions of lateral support (which determines the effective length of beams). For girders an additional parameter is added, i.e. the number of point loads (imposed by the beams) and thus the spacing of the loads (beam spacing).

Field work was conducted to identify the popular I-sections in industry as well as the popularity of various design parameters (frequency of occurrence of different load intensities, etc.). This information was used to determine weighting factors to account for the popularity of design parameters in practice (refer to Section 5.5).

The correct way to conduct the field work would be to do a comprehensive survey of all the applications of I-sections for beams, girders, columns, rafters and other members in the country, covering all types of structures. Data on the historical use of the different available I-sections would also be useful. However, data on historical use could not be obtained (regarded as trade secrets) and a comprehensive survey would have taken too long and been too expensive. It was thus decided to do a survey amongst a limited number of engineers and to focus more on the development of the optimisation methodology (refer to Chapter 5) rather than to produce a final optimal set of I-sections for practice.

The field work and survey took the form of interviews rather than questionnaires, because engineers find it hard to identify the popular design parameters immediately and they tend to believe that the distribution is irregular. An investigator conducting an interview can help the engineer through the process of analysing the relevant information, to obtain the popular design parameters.

Engineers use I-sections for columns, beams (secondary beams), girders (primary beams), portal frames, roof structures (rafters), etc. It is difficult to say how much they are used for all the purposes mentioned, but only beams and girders were considered in this research project

(Section 1.4).

The definitions for GIRDERS and BEAMS, which are also referred to as primary and secondary beams, sometimes also differ from one engineer to the next. These terms are defined in Section 1.4 to eliminate any misunderstanding or uncertainty.

It was interesting to find during the interviews that the assumptions made by engineers can differ a great deal from one engineer to the next. These assumptions are also reflected in the popularity of the various design parameters.

7.2 Background of engineers interviewed

One steel fabricator (Appendix B) and four structural engineers (Appendix C) were interviewed. Although the interview with the steel fabricator focused more on the practical considerations of fabricating welded I-sections, information was also obtained relevant to which I-sections are used more frequently in practice.

All the engineers interviewed had extensive experience in steel structures and, being from different companies, provided a broader picture of which I-sections and design parameters occur most often in practice. Table 7.1 provides background on the structures the participants typically design and how much experience each has.

Table 7.1: Background of participant engineers

Participants	**1**	**2**	**3**	**4**
Position at company	Director	Design Engineer	Technical Director	Design Engineer
Academic qualifications	BEng MEng PrEng	B.Sc. (Eng) M.Sc. (Eng) GDE PrEng	MEng PrEng	BEng MEng PrEng
Experience (years)	23	7	37	10
Structures designed by participant	Industrial structures	Industrial structures, warehouses, shopping centres and office blocks.	Light industrial structures (Warehouses)	Industrial structures, office blocks, flat blocks and houses.
Number of projects per year	400 (300 steel structures and 50 large structures)	30 to 40 (cost ranging from R1 million to R 1 billion)	6 (normally in the range of 30 000 m^2)	23 to 25 (20 houses, 1 to 2 industrial structures and 2 to 3 office or flat blocks)
Number of structures designed per year	50 (large structures)	30 to 40	6	23 to 25

From the information presented in Table 7.1 and Appendix C it was concluded that Participants 1 and 2 have more experience in specifying steel I-sections than Participants 3 and 4. The structures they design also incorporate more I-sections than those of Participants 3 and 4. The information from Participants 1 and 2 was thus held in higher regard.

7.3 Findings of survey

7.3.1 Currently popular I-sections

Each engineer has different preferences according to which I-sections are often specified for beams and girders. The reasons for these preferences are normally related to the typical structures the engineer designs. To minimise cost, different assumptions are made in the design proses (refer to Section 7.3.6). Sometimes the preferences reflect the views of the leadership in a company and their experience. In some cases the rules are even prescribed by clients.

The steel fabricator is referred to as Participant 5 in this section. The interview with the steel fabricator only contributed to this section of Chapter 7.

7.3.1.1 Beams

The following table contains the I-sections that are frequently used by all the participants and shows which sections are particularly popular with each participant (marked with an X).

Table 7.2: The popular I-sections used for beams

	Participants				
I-sections	**1**	**2**	**3**	**4**	**5**
IPE_{AA} 160		X			
IPE 160		X			
IPE_{AA} 200				X	
IPE 200		X		X	
UB 203 x 133 x 25	X		X		
UB 254 x 146 x 31			X	X	X
UB 305 x 102 x 25		X	X	X	
UB 305 x 165 x 40				X	
UB 356 x 171 x 45			X		
UB 406 x 140 x 39		X	X		
UB 406 x 178 x 54			X		
UB 457 x 191 x 67		X			

Sometimes the size of an I-section has a larger influence on why it is being used than its capacity. Participant 4, for example, had different groups of I-sections for houses and industrial structures. For houses, 200 IPE's and 254 mm deep I-sections are normally used. One of the reasons why the 254 mm deep I-sections are used, is because the brick layers fit well into the space between its flanges. The larger 305 mm and 356 mm deep I-sections, on the other hand, are used more often in industrial structures, as the beams normally have larger spans in such structures (refer to Appendix C).

7.3.1.2 Girders

Table 7.3 presents all the typical I-sections used for girders by the participants.

Table 7.3: The popular I-sections used for girders

	Participants				
I-sections	**1**	**2**	**3**	**4**	**5**
UB 305 x 165 x 46	X				
UB 406 x 140 x 39		X			
UB 457 x 191 x 67		X			
UB 533 x 210 x 82		X			
Plate girders	X	X			
Do not use girders			X	X	

As shown in Table 7.3, Participants 3 and 4 do not use girders, as their beams are normally directly supported by columns. Participants 1 and 2 do however use girders in their structures. No typical girder sections were obtained from Participant 5.

Participant 1 normally uses 305 x 165 x 46 I-sections, and welded plate girders only about 10 % of the time. The reason given was that welded plate girders are normally more expensive than hot-rolled I-sections. When the participant needs to use a plate girder, the plate girders specified in the Red Book (SAISC, 2013) are not used, because they are too deep.

The girders used by Participant 2, on the other hand, are normally larger than the sections used by Participant 1, because of higher design loads (refer to Section 7.3.4). Participant 2 does however use the plate girders specified in the Red Book (SAISC, 2013), because space limitations on girders are not really of concern in his designs.

7.3.2 Floor systems used in practice

There are mainly six different floor systems used in practice, namely steel grating, Vastrap (steel plate), in-situ concrete slabs, hollow core floors and concrete floors on metal deck with or without shear studs. Concrete floors with shear studs behave compositely with steel beams and those without, do not.

7.3.2.1 Grating

Grating, also known as open-grid flooring, is an economical lightweight system that permits the flow of air and the passage of light to the areas below (Refer to Figure 7.1). Grating is virtually self-cleaning, as it does not allow rubble or dust to accumulate on it, but allows water and oil to drain easily through it. Although grating has a lot of advantages, its resistance to shear loads in its own plane is low and it can therefore not be considered as bracing (lateral support) for the beams supporting the floor system, according to the Red Book (SAISC, 2013). However, according to Participant 2, grating can provide lateral support (refer to Appendix C).

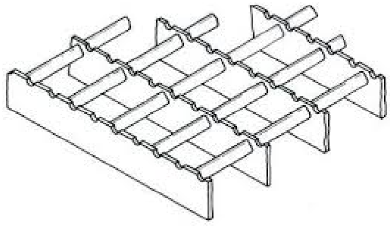


Figure 7.1: Grating (Grating World (Pty) Ltd, 2013)

Grating is limited to spans of about 1.5 m for 25 x 3 grating and 2.5 m for 40 x 4.5 grating, according to Vital Engineering & Angus McLeod (2016) and SAISC (2013). Grating can withstand loads of up to 186.34 kPa, but are normally only used for loads up to 7.5 kPa (heavy duty walkways). More information about grating can be found in Vital Engineering & Angus McLeod (2016) and SAISC (2013).

7.3.2.2 Vastrap

Vastrap is a solid plate floor and is commonly used where spillage, such as chemicals, needs to be contained or where a high level of hygiene is required. Vastrap also has considerable stiffness and shear resistance in its own plane when it is properly connected to the supporting beams. Vastrap can thus be employed as secondary bracing, providing lateral support to the supporting beams (SAISC, 2013).

Vastrap can span from 0.6 m to 2 m and is normally manufactured in a standard width of 1.2 m (SAISC, 2013). Refer to SAISC (2013) for more information about Vastrap.

7.3.2.3 Reinforced concrete floors

According to Participant 1, normal concrete (in-situ concrete) floors are constructed with the use of shuttering on I-sections. The thicknesses of these floors range from 175 mm to 225 mm (refer to Appendix C). Many concrete floors use metal deck as permanent shuttering.

Two different deck systems can be used as shuttering, namely Bond-dek and Bond-lok (refer to Figure 7.2). Both systems are basically for one-way slabs designed for the purpose of carrying uniformly distributed loads. These floor systems should therefore be checked when they are subjected to concentrated loads or moving loads (SAISC, 2013).

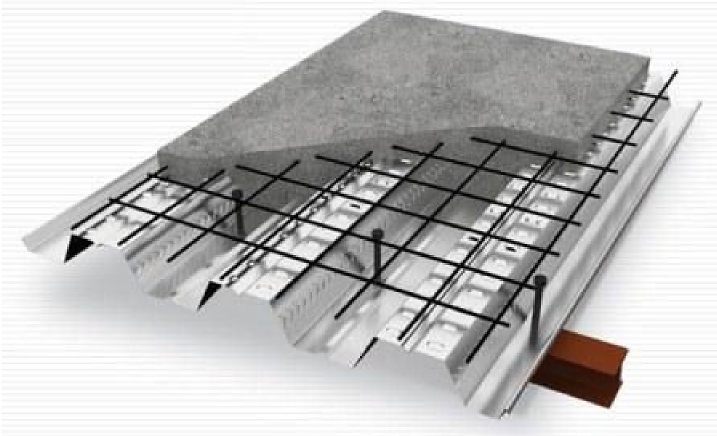


Figure 7.2: Concrete floor on metal deck (Www.dianliwenmi.com, 2016)

Bond-dek slabs can span between 2 m and 4.5 m depending on the slab thickness and Bond-dek thickness. The overall slab thickness of Bond-dek slabs can range from 140 mm to 250 mm and the nominal Bond-dek thicknesses between 0.8 mm and 1.2 mm (SAISC, 2013).

The capacity of Bond-dek slabs ranges from 11.21 kN/m^2 to 23.21 kN/m^2, with nominal permanent loads ranging from 2.52 kN/m^2 to 5.15 kN/m^2 (SAISC, 2013). More information about Bond-dek slabs can be obtained from SAISC (2013) or GRS Global Roofing Solutions (2016).

Bond-lok slabs can span a lot further than Bond-dek slabs, provided that they are propped during construction, with span between 1.5 m and 7.3 m depending on the slab thickness and Bond-lok thickness. The slab thickness of Bond-lok slabs range from 100 mm to 250 mm (SAISC, 2013).

The capacity of Bond-lok slabs ranges from 10.3 kN/m^2 to 69.9 kN/m^2, with nominal permanent loads ranging from 2.48 kN/m^2 to 6.09 kN/m^2 (SAISC, 2013). More information about Bond-lok slabs can be obtained from SAISC (2013) or GRS Global Roofing Solutions (2016).

7.3.2.4 Hollow core concrete floors

Hollow core floors are lighter than solid concrete floors, because of the circular holes in them (refer to Figure 7.3). These floors are popular in office buildings and houses. They are available in thicknesses from 150 mm to 250 mm (TOPFLOOR, 2016a), with nominal permanent loads ranging from 263 kg/m^3 to 371 kg/m^3 (TOPFLOOR, 2016b). Hollow core floors can span up to 13 m, depending on the wire spacing, slab thickness and applied load.

Figure 7.3: Hollow core floor (BISON, 2007)

7.3.3 Popularity of floor systems

From the four participants, information was obtained covering four different types of structures (refer to Appendix C). These structures include industrial structures, light industrial structures, office blocks and houses. Houses were however not considered in this section, because I-sections are seldom used in them.

7.3.3.1 Industrial structures

Table 7.4 presents the different floors systems used in industrial structures, with their typical thicknesses and corresponding popularity percentages. Only Participants 1, 2 and 4 have experience in industrial structures.

As shown in Table 7.4, Participant 2 categorised industrial structures into two different groups: Group 1 referring to industrial structures with large work areas and Group 2 including industrial structures with large machinery areas.

As mentioned in the Red Book (SAISC, 2013) and shown in Table 7.4, metal flooring such as grating and Vastrap is used extensively in industrial structures (mining industry, process and chemical plants, etc.).

Grating is normally more popular than Vastrap, as it is easier to construct. According to Participant 1, it is also cheaper for the same reason (refer to Appendix C). Vastrap is therefore only used when it is really needed, such as when dust or waste is not allowed to flow to the lower floors, etc.

Concrete floors are not used often by Participant 1, because of its longer construction time and higher permanent loads. The participant does not use hollow core slabs either, citing their low vibration resistance and stiffness.

Participant 2 uses a lot of grating, but for work areas composite concrete floors are used more, due to vibration constraints. The floors in machinery areas do not have the same vibration constraints as in work areas and for this reason tend to consist of grating and Vastrap. The concrete floors of Participant 2 are normally constructed with shear studs to form composite beams, as some floors need to withstand high loads in industrial structures, especially in storage areas. Concrete floors on metal deck without shear studs are frequently used by Participant 2 when there are plans to expand the structure, as they are easier to replace when another floor is required.

Participant 4, on the other hand, uses both hollow core and in-situ floors, as the structures designed by the participant do not have the same vibration constraints as required for the industrial structures of Participants 1 and 2 (refer to Appendix C).

Table 7.4: Popularity of floor systems in industrial structures (%)

		Participants			
		1	**2**		**4**
Floor systems	**Thickness range (mm)**	**Structures**			
		Industrial structures	Industrial structures (Group 1)	Industrial structures (Group 2)	Industrial structures
Grating	30 x 4.5 40 x 3	90 (30 x 4.5)	40	50	
Vastrap	4.5 - 6	9		50 (6 x 8)	
Concrete on metal deck (not composite)	200	1			
Concrete on metal deck (composite)	170 - 200 270 (30 kPa)		60		
Hollow core concrete	170 - 255				50
In-situ concrete	170 - 255				50

7.3.3.2 Light industrial structures

Light industrial structures can also be defined as warehouses. These light industrial structures do however have office floors, which typical warehouses do not have.

Table 7.5 presents the popularity of the floor systems used by Participant 3, as the other participants do not have any experience in light industrial structures. The concrete floors in these structures are normally Bond-dek floors, because they are easy to construct and can provide composite action when needed (refer to Appendix C).

Table 7.5: Popularity of floor systems in light industrial structures

Floor systems	**Thickness range (mm)**	**Popularity (%)**
Grating	40	20
Vastrap	4.5	20
Concrete on metal deck (not composite)	140	20
Concrete on metal deck (composite)	140	40

7.3.3.3 Houses, office blocks and apartment buildings

Participants 1, 2 and 4 provided information on floor systems for houses; office blocks and apartment buildings (refer to Appendix C). However, only Participants 2 and 4 design these structures on a regular basis. Table 7.6 presents the popularity of the different floor systems in these structures.

Table 7.6: Popularity of floor systems in houses, office blocks and apartment buildings (%)

		Participants	
		2	**4**
Floor systems	**Thickness range (mm)**	**Structures**	
		Office blocks	Office blocks, apartment buildings and houses
Concrete on metal deck (composite)	140 - 170	5 - 10	
Hollow core concrete	170 - 255		50
In-situ concrete	170 - 255	90 - 95 (140 - 170)	50 (170 - 255)

Participant 1 mentioned in the interview that hollow core floors are more popular in houses than in industrial structures and that such concrete floors normally range between 170 mm and 255 mm, which corresponds with 2 to 3 brick layers and the slab range of Participant 4 (refer to Appendix C).

In-situ concrete floors in office buildings are also normally constructed with permanent shuttering when used on I-sections, for the same reasons as mentioned in Section 7.3.3.1.

Based on the interviews (refer to Appendix C), it was found that I-sections are not often used in apartment buildings and houses. These buildings were therefore disregarded in the weighting factors discussed in Sections 7.4 and 7.5.

7.3.4 Typical loads

7.3.4.1 Typical permanent loads

The permanent loads carried by I-beams are normally a combination of the weight of the supported floor and the own weight of the beam supporting the floor. When concrete floors are used in office blocks, the own weight of a 50 mm screed should also typically be added. In industrial buildings screed is normally not added. This was reported by all the participants (refer to Appendix C).

The permanent loads of Participant 3 are usually in the region of 3 kPa, all included.

Participant 4 also considers extra permanent loads on the I-beams, like brick walls with permanent loads of approximately 18 kN/m. This distributed line load can be converted into a uniformly distributed load (kPa) when the walls are not located directly above the beams.

The permanent load on a girder is the sum of the own weight of the girder and the point loads applied by the beams supporting the floors.

7.3.4.2 Typical imposed loads

Imposed loads normally correspond to the prescribed imposed loads in the loading code (SABS, 2011b), but can be a lot higher in highly loaded structures, such as industrial structures. Table 7.7 presents the typical imposed loads for the structures designed by the participants.

The X marks show which imposed loads were reported being used by a participant. It also shows if the imposed loads are in line with what is specified in SANS 10160-1 (SABS, 2011b).

As shown in Table 7.7, Participant 2 did not have additional imposed loads on his structures, as all the imposed loads are included in one load.

Some of the imposed loads in Table 7.7 are much higher than the specified loads in the loading code (SABS, 2011b). The loads in the industrial storage areas and industrial rebuild areas of Participant 2 are, for example, a lot higher than the minimum of 5 kPa specified in the loading code (SABS, 2011b). The reason is that storage areas need to withstand higher loads than what is normally specified for industrial structures. Rebuild areas are over-designed because of the uncertainty over the future loads that might be applied to the structure. The industrial flat roofs of Participant 2 are also designed for higher loads than normal, as they are regarded as potential storage space (5 kPa in the loading code) and thus designed accordingly.

Table 7.7: Typical imposed loads in practice

Parts of structure	Live loads (kPa)	Other imposed loads (kPa)	Participants 1	2	3	4	Loading code (SANS 10160-1)
Industrial floors	5	1 (services)	X				X
Industrial platforms (floors)	5			X	X		X
Industrial storage areas (floors) (1)	5	1 - 1.5 (services)				X	X
Industrial storage areas (floors) (2)	10			X			X (load > 5 kPa)
Industrial rebuild areas (floors)	20			X			
Industrial roofs	1	0.5 (services)	X		X		X (0.75 kPa)
Industrial flat roofs	5			X			X (load > 5 kPa)
Office floors	3			X			X (2.5 kPa)
Flat roofs (Office and flat blocks)	1.5 - 3					X	X (2 kPa)

7.3.4.3 Typical girder loads

Point loads are applied to girders by the beams they support. According to Participant 1, these loads are normally between 30 kN and 70 kN, with point loads between 30 kN and 50 kN occurring most often. However, according to Participant 2 these point loads can range anywhere between 10 kN and 2000 kN, with spacing according to the beam spacing. The other participants could not provide any information on the topic, as they do not use girders often.

7.3.5 Popularity of spans

The typical I-section spans vary from one engineer to the next. Normally column positions are specified by the engineer for industrial structures and by the architect for offices, houses and apartment buildings. This makes it difficult to determine how popular a certain span or range of spans would be. Most of the participants could however say which spans or span ranges occur most in their structures.

Participant 1 uses beam spans ranging between 2 m and 4 m, with a typical maximum span of 3 m. Girders, on the other hand, would normally span 5 m, but can span up to 20 m in the structures the participant designs.

Participant 2 was the only participant who could provide the approximate popularity of certain span ranges, and these are presented in Table 7.8. The popularity percentages are for beams and girders in industrial and office buildings.

Table 7.8: Popularity of beam and girder spans according to Participant 2

Beam span ranges (m)	**Percentage in industrial structures**	**Percentage in office blocks**
0 - 2	10	20
2 - 4	30	30
4 - 6	30	30
6 - 8	20	15
8 - 10	10	5

Participant 3 normally uses spans of 7.8, 8.4, 8.5 and 10 m for beams and girders, but does not use girders often. These spans correspond to the specifications for parking grids (refer to Appendix C).

Participant 4 normally uses spans that are a lot larger than what is used by the other participants, but the loads tend to be lower in the structures the participant designs. The beam spans normally range between 6 m and 10 m in houses and between 5 m and 15 m in industrial structures designed by the participant. Spans of 5 m are normally used in small industrial buildings and 15 m spans for larger industrial buildings. The participant could not provide girder spans, as the participant does not use girders much.

7.3.6 Popularity of support conditions

The support conditions of girders and beams can be broken up into two categories, the first being the support conditions at the ends of the beam or girder (fixed or pinned) and the second, the lateral support condition of the compression flange of the section (laterally supported or unsupported).

7.3.6.1 End support conditions

The participants normally design their beams and girders as pinned, as shown in Table 7.9. This information supports the decision to limit this study to simply supported beams.

Table 7.9: Popularity of pinned and fixed support conditions

	Beams		Girders	
Participants	**Percentage pinned**	**Percentage fixed**	**Percentage pinned**	**Percentage fixed**
Participant 1	80	20	80	20
Participant 2	90 - 95	5 - 10	100	0
Participant 3	90 - 95	5 - 10	100	0
Participant 4	80	20	No value	No value

7.3.6.2 Lateral support conditions

Table 7.10 shows the popularity of the different support conditions in structures designed by all the participants.

Table 7.10: Popularity of lateral support conditions

	Beams		Girders	
Participants	**Percentage laterally supported**	**Percentage laterally unsupported**	**Percentage laterally supported**	**Percentage laterally unsupported**
Participant 1	5	95	0	100
Participant 2	70	30	60	40
Participant 3	80	20	100 (if used)	0
Participant 4	20	80	No value	No value

The assumptions regarding the lateral support of flanges varies from one participant to the next.

Participant 1, for example, normally considers beams as laterally unsupported in the design process, because these beams typically support grating and sometimes concrete floors on metal deck, without shear studs. Any grating is attached by a method that does not provide lateral support (refer to Appendix C). The beams of Participant 1 therefore tend to be bigger than those of the other participants, who design their beams as laterally supported.

Participant 2, on the other hand, assumes that grating provides lateral support. This assumption makes it possible to use lighter I-sections than those of Participant 1. According to this participant, there is also a paper available from the University of the Witwatersrand which supports and proves this assumption. The participant also uses different groups of I-sections when designing composite beams or platforms (refer to Appendix C).

Participant 3 has a different assumption when it comes to concrete floors on metal deck. The participant assumes that these concrete floors provide lateral support, even when used without shear studs. The participant however always ensures that the metal deck of the concrete floor is welded to the beams in order to ensure lateral support (refer to Appendix C).

7.3.7 Typical effective lengths

The effective length of beams and girders are dependent on the lateral support conditions of the section. Beams will have an effective length equal to their span if they are laterally unsupported, but when laterally supported they have an effective length of zero.

Girders also have an effective length of zero when they are laterally supported, but this is mostly not the case. When girders are laterally unsupported, which is normally the case, their effective length is equal to the beam spacing when the beams are supported in its longitudinal axis. When the beams are not effectively supported, the effective length of a girder is equal to the span of the floor bracing.

Participants 1 and 4 do not normally assume that the beams supported by girders provide lateral support. This means that Participant 1 normally assumes that the effective length of a beam is equal to its span and that the effective length of a girder is equal to the distance between floor bracing points. The beams are normally braced in 3 m (60 % of the time) to 5 m (40 % of the time) bays in order to shorten the effective length of the girders (refer to Appendix C). Participant 4 could not provide floor bracing spans, as the participant does not use girders often.

The floors of Participants 2 and 3, on the other hand, are normally assumed to provide lateral support. The beam effective lengths of these participants are therefore normally equal to zero and the girder effective lengths equal to the beam spacing (refer to Section 7.3.8).

7.3.8 Beam spacing

The beam spacing is normally dependent on the allowable span of the floor system used.

The beam spacing used by Participant 1 is typically in the range of 1.5 m, which corresponds to the typical allowable span for grating. The beam spacing of Participant 2 is normally between 1.5 m and 2 m, which corresponds to allowable grating and minimum Bond-dek spans. The beams of Participant 3 are normally spaced between 2.5 m and 2.8 m, which corresponds to the span range of unpropped Bond-dek (refer to Appendix C).

7.4 Popularity weighting factors for BEAM conditions

A simply supported beam's configuration depends on two properties, its span and the lateral support to its compression flange, which determines its effective length and thus its resistance to lateral torsional buckling. Laterally supported beams have an assumed effective length of zero and laterally unsupported beams an assumed effective length equal to its span. Beam loading was assumed to be uniformly distributed, as they commonly support floors.

Two different sets of weighting factors were determined for different combinations of distributed load and beam conditions, namely Weighting A and B.

The first set, Weighting A ("Preliminary weighting factors for beams"), was determined rather crudely at the start of the research project, by simply thinking what might be the frequency of occurrence of different spans and loadings in practice. Thus it was based on intuition rather than real data.

The second set, Weighting B ("Weighting factors for beams based on survey"), was based on the information obtained from interviews with engineers, as discussed in Section 7.3.

Weighting B should yield better results than Weighting A, as the weighting factors of Weighting B was based on survey data which corresponds directly to practice. Weighting A was nevertheless retained as it allows for a sensitivity analysis of the weighting factors.

The weighting factors of Weighting A correspond to Design Space A and Weighting B to Design Space B.

7.4.1 Weighting A - Preliminary weighting factors for beams

Weighting A specifies popularity weighting factors for 2686 different data points, defined by beam span, uniformly distributed load and lateral support (refer to Section 5.3.1). The beam spans range from 2 m to 10 m and the uniformly distributed loads from 1 kN/m to 40 kN/m.

7.4.1.1 Weighting factors for lateral support conditions

In Weighting A, lateral supported and unsupported conditions were assumed to occur equally as often. The lateral support weighting factors ($f_{support}$) for laterally supported and unsupported sections were therefore both equal to 1.0, which means that lateral support conditions were effectively not taken into account.

7.4.1.2 Weighting factors for span

The assumed span weighting factors (f_{span}) of Weighting A were distributed as shown in Figure 7.4. Spans of between 4 m and 8 m were considered to be the most popular, dropping off linearly on one side to a minimum span of 2 m and on the other to a maximum span of 10 m.

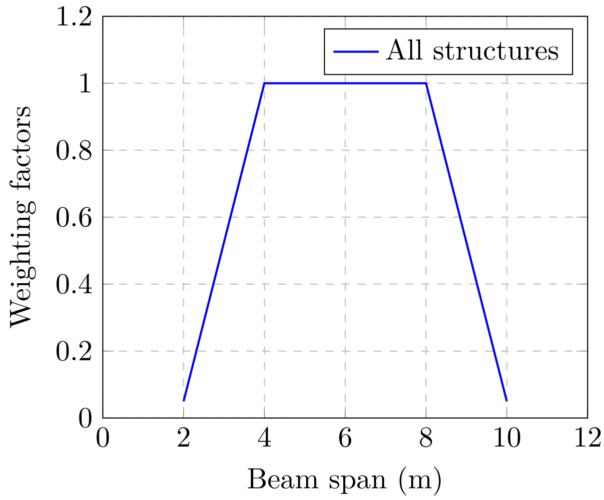


Figure 7.4: Beam span weighting factors of Weighting A (f_{span})

7.4.1.3 Weighting factors for loading

The load weighting factors (f_{load}) accounted for the loading in two different types of structures, office buildings and industrial structures. The load weighting factors for these structure types, with all the loads below unfactored, were determined as described hereunder.

In **office buildings** with a typical 140 mm concrete floor on metal deck, the permanent load is approximately 2.5 kPa, according to p.14.24 - p.14.25 of the Red Book (SAISC, 2013). The permanent load of a 150 mm thick hollow-core slab is also approximately 2.5 kPa without screed, according to Topfloor's website (TOPFLOOR, 2016b). A 50 mm levelling screed will add another 1 kPa to the permanent load. The permanent load of office buildings is therefore normally around 3.5 kPa.

The imposed load in an office building is also typically 2.5 kPa, according to SABS (2011b), with values up to 10 kPa in storage areas. An extra 0.5 kPa needs to be allowed for services, adding up to a total imposed load of about 3 kPa.

Where the spacing of the beams ranges between 2 m and 3 m, with spacing associated with hollow core concrete slabs ranging up to 6 m and more, the weighting factors presented in Figure 7.5 were suggested for office buildings. The uniformly distributed loads presented in Figure 7.5 are ultimate limit state loads, that were factored with load factors of 1.2 for permanent loads and 1.6 for imposed loads, as specified by SANS 10160-1 (SABS, 2011a).

In **industrial structures**, steel flooring in the form of grating or plate (e.g. Vastrap) is commonly used, with a permanent load between 0.4 kPa and 0.5 kPa, according to Table 2.23 of the Red Book (SAISC, 2013). Hollow-core flooring of 200 mm can also be used, with a permanent load of 4 kPa (levelling screed included), according to Topfloor's website (TOPFLOOR, 2016b).

The imposed loads of industrial structures can vary hugely, depending on the storage or equipment they need to support. Many areas are also subjected to live loads of between 3 kPa and

5 kPa, with some in the range of 10 kPa (SABS, 2011b). Provision also has to be made for services of about 0.5 kPa.

The beam spacing for steel flooring in industrial structures varies from 1 to 2 m and for hollow core flooring from 1 to 2.5 m.

After accounting for the loads, beams spacing and associated load factors, the distribution of weighting factors presented in Figure 7.5 for industrial structures was adopted.

The weighting factors for office buildings and industrial structures were combined on a 50/50 basis to obtain the final load weighting factors, as shown on Figure 7.5.

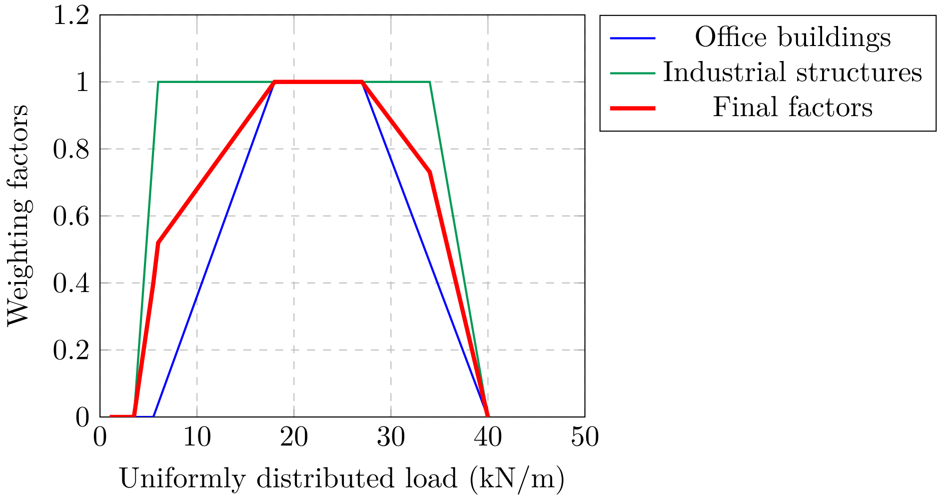


Figure 7.5: Load weighting factors of Weighting A (f_{load})

7.4.2 Weighting B - Weighting factors for beams based on survey

Weighting B specifies popularity weighting factors for 3560 different data points (refer to Section 5.3.1), with spans ranging from 0.5 m to 10 m and factored uniformly distributed loads from 1 kN/m to 45 kN/m, including different lateral support conditions.

7.4.2.1 Weighting factors for lateral support conditions

The popularity of lateral support conditions was used in combination with the number of structures designed by each participant to calculate the weighting factors for lateral support conditions ($f_{support}$). The popularity of lateral support conditions are presented in Table 7.10 and the number of structures designed by the respective participants in Table 7.1.

A total of 101 structures were considered in the process of determining the weighting factors. The popularity of laterally supported beams was found to be 35.94 % and that of laterally unsupported beams 64.06 %, with weighting factors of 0.36 and 0.64 respectively. The popularities of the laterally supported and unsupported conditions were calculated as presented in Table D.1 of Appendix D.

7.4.2.2 Weighting factors for span

The span weighting factors (f_{span}) of Weighting B were determined by using the popular beam spans and their popularity percentages from Participants 1, 2 and 3 (refer to Section 7.3.5) in combination with the number of structures designed by each participant, as presented in Table 7.1. The most popular span range of each type of structure received a weighting factor of 1.0, while the rest of the weighting factors were calculated according to the most popular span range and the popularity percentages of each span range, as presented in Figure 7.6. These weighting factors were then combined using the frequency of occurrence of each type of structure considered, as presented in Table 7.11, to yield the "Combined factors" as presented in Figure 7.7. Refer to Appendix D for the calculations.

Table 7.11: Number of structures considered for span weighting factors of Weighting B

Structures	**Participants**	**Number of structures**	**Frequency of occurrence (%)**
Industrial	1	50	52.08
	2	30	31.25
Office	2	10	10.42
Light industrial	3	6	6.25

Participant 2 designs between 30 to 40 structures per year, being a combination of office and industrial structures, with the assumption that 30 of these structures are industrial structures and 10 office blocks, as shown in Table 7.11. This assumption was made as the participant designs industrial structures more frequently than other structures.

The information from Participant 4 was excluded from the span weighting factors, as it was insufficient to use.

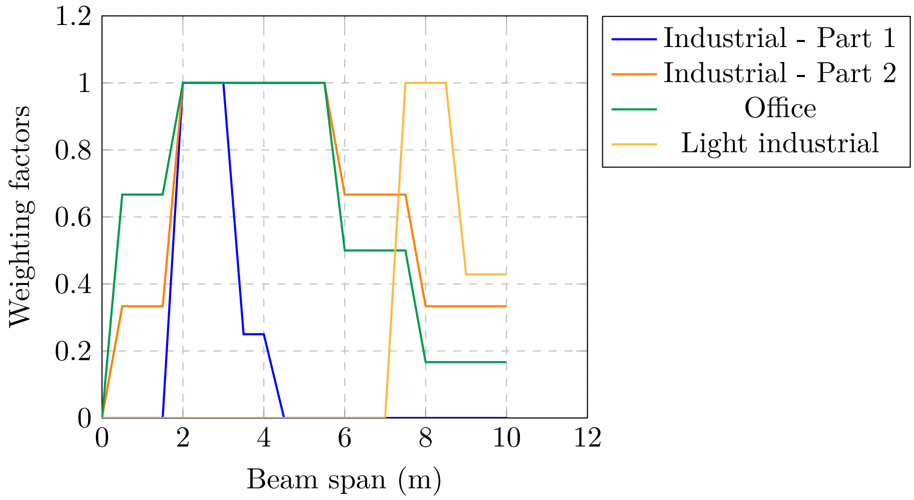


Figure 7.6: Beam span weighting factors of the different types of structures

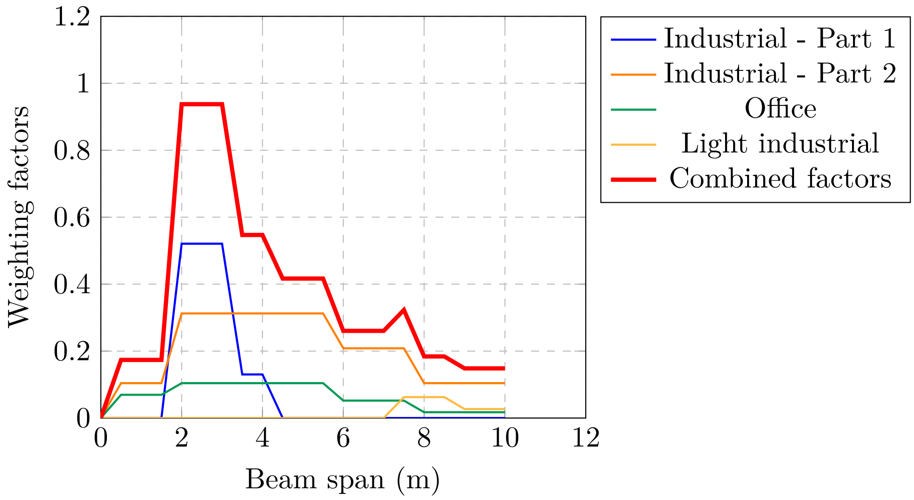


Figure 7.7: Beam span weighting factors of Weighting B (f_{span})

Spans of between 2 and 3 m were found to be the most popular, dropping off on both sides to a minimum span of 0.5 m and a maximum span of 10 m, as shown on Figure 7.7. The "Combined factors" were smoothed out to form the "Practical factors", represented by the red line on Figure 7.8. These "Practical factors" were used in the optimisation process.

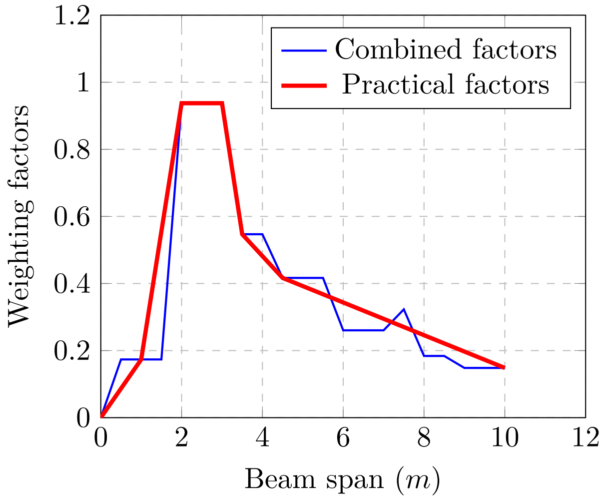


Figure 7.8: Final beam span weighting factors of Set B (f_{span})

7.4.2.3 Weighting factors for loading

Different ranges of load weighting factors (f_{load}) were first developed for the three different structures, i.e. industrial structures, light industrial structures and office buildings. These weighting factors were then combined according to the frequency of occurrence of each structure to form the final load weighting factor, which were used in the optimisation process.

All of the loads below are unfactored, unless they are referred to as ultimate loads. Load factors of 1.2 and 1.6 were respectively used for permanent and imposed loads to calculate the ultimate loads.

A load weighting factor of 1.0 was always adopted for the most popular distributed loads, dropping off to a 0.0 weighting factor for the minimum and maximum distributed loads that can conceivably occur.

Industrial structures

The information from Participants 1 and 2, discussed in Section 7.3.3.1, was considered in the load weighting factors for industrial structures. The number of industrial structures designed by Participant 4 was much less than those designed by Participant 1 and 2. The information from Participant 4 was thus not considered for these weighting factors.

Only two different loads were considered in the process of determining the load weighting factors, namely permanent loads and imposed loads.

The imposed loads on industrial structures normally range between 5 kPa and 6 kPa (5 kPa plus 1 kPa for services), according to the participants (refer to Section 7.3.4.2). Permanent loads, on the other hand, vary significantly depending on which floor system is used. Three different floors systems were considered for industrial structures, as identified by the participants, namely grating, Vastrap and concrete on metal deck. The popularities of these floor systems, from Section 7.3.3.1, were used in combination with the number of structures designed by the participants per year to determine the popularity of the floor systems over the range of industrial structures (refer to Table 7.11). The popularities of the floor systems are presented in the following table.

Table 7.12: Popularity of industrial floor systems

Floor systems	**Popularity (%)**
Grating	73.13
Vastrap	15.00
Concrete on metal deck	11.87

According to the participants, 30 x 4.5 and 40 x 3 **grating** is commonly used in industrial structures, with a span of 1.5 m (refer to Section 7.3.3.1). It was however found that the permanent loads of these grating sizes, range from 0.33 kPa to 0.36 kPa (SAISC, 2013) and that the specified imposed loads were too high for 30 x 4.5 grating with a span of 1.5 m (refer to p.14.43 of the Red Book (SAISC, 2013)). To correct this, a span of 1.2 m was used to calculate the popular ultimate distributed loads on 30 x 4.5 grating.

The minimum and maximum ultimate distributed loads of grating are shown in Table 7.13, as obtained from p.14.43 of the Red Book (SAISC, 2013). The minimum distributed load of grating was assumed to be the ultimate distributed load of 25 x 3 grating at a span of 1.5 m, with the maximum distributed load assumed to be the ultimate distribute load of 40 x 4.5 grating at a span of 1.25 m. The load weighting factors for grating are represented by the blue line on Figure 7.9.

Vastrap is available in thicknesses of 3 mm to 8 mm and standard widths of 1.2 m, but according to the participants 4.5 mm and 6 mm are the most popular in industrial structures. It was also found that these Vastrap sizes can only carry the ultimate distributed loads up to a span of 1.2 m, if they are pinned. A 1.2 m maximum span was thus assumed for the calculations. The factored popular distributed loads can be found in Table 7.13. These loads were calculated with the permanent loads of the popular Vastrap sizes, which are 0.37 kPa to 0.49 kPa according to Table 2.23 of the Red Book (SAISC, 2013).

The maximum and minimum distributed loads of Vastrap, as shown in Table 7.13, were calculated with the use of Section 12.2.5 of the Red Book (SAISC, 2013). The minimum distributed load was assumed to be the capacity of a 3 mm Vastrap plate with a span of 1.6 m and the maximum distributed load, the capacity of a 8 mm Vastrap plate with a span of 1.4 m. The load weighting factors for Vastrap are represented by the orange line on Figure 7.9.

According to the participants, 170 mm to 200 mm thick **concrete floors on metal deck** are typically used with floor spans of 2 m in industrial structures. The permanent load of these floors range between 3.22 kPa and 3.97 kPa, according to p.14.24 to p.14.25 of the Red Book (SAISC, 2013). The combination of the 2 m span with the factored imposed and permanent loads yielded popular distributed loads as shown in Table 7.13.

Although the popular thicknesses ranged between 170 mm and 200 mm, concrete floors on metal deck can have thicknesses ranging from 140 mm to 250 mm. The minimum load shown in Table 7.13, is the own weight of a 140 mm thick slab with a span of 2 m. The maximum load is the capacity of a 250 mm thick slab with a span of 2 m. The information was obtained from p.14.24 to p.14.25 of the Red Book (SAISC, 2013). The adopted load weighting factors for concrete floors on metal deck are demonstrated by the green line on Figure 7.9.

Table 7.13: Range of ultimate distributed loads in industrial structures

Floor systems	**Minimum load (kN/m)**	**Popular load range (kN/m)**	**Maximum load (kN/m)**	**Span range considered (m)**
Grating	4.0	10.5 - 15	35.0	1.25 - 1.5
Vastrap	3.3	10.1 - 12.2	23.0	1.2 - 1.6
Concrete on metal deck	6.0	23.7 - 28.7	46.6	2.0

Each range of weighting factors were factored according to the corresponding popularity percentage of each floor system (as presented in Table 7.12) and then added together to form the combined weighting factor distribution for industrial structures, as presented in Figure 7.9 by the red line.

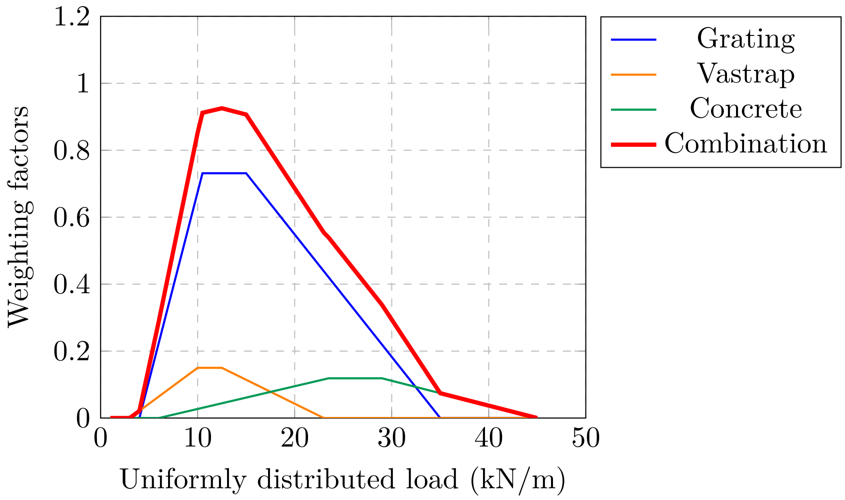


Figure 7.9: Load weighting factors for industrial structures of Weighting B (f_{load})

Light industrial structures

Only Participant 3 had experience in light industrial structures and therefore only the information from Participant 3 was considered in the development of the load weighting factors for light industrial structures (refer to Section 7.3.3.2),

According to Participant 3, the imposed loads on light industrial structures are normally 5 kPa (refer to Section 7.3.4.2), with the permanent loads normally the own weight of one of the following floor systems: grating, Vastrap or concrete floors on a metal deck (the same as industrial structure). The popularities of these floor systems are presented in Table 7.14.

Table 7.14: Popularity of light industrial floor systems

Floor systems	**Popularity (%)**
Grating	20
Vastrap	20
Concrete on metal deck	60

A 40 x 3 **grating** with a span of 1.5 m (the same as industrial structures) is normally used in light industrial structures. A 40 x 3 grating has a permanent load of 0.33 kPa (refer to the Red Book (SAISC, 2013)). The combination of the spacing with the factored imposed and permanent loads yielded factored popular distributed loads as shown in Table 7.15.

The maximum and minimum distributed loads used in Figure 7.10 for grating are more or less the same as for industrial structures. The only difference being, that the capacity of 40 x 4.5 grating with a span of 1.5 m was used instead of a span of 1.25 m. Refer to p.14.43 of the Red Book (SAISC, 2013) for more information. The adopted load weighting factors for grating are demonstrated by the blue line in Figure 7.10.

A **Vastrap** thickness of 4.5 mm in standard widths of 1.2 m over a span of 1.5 m is normally used by Participant 3. Vastrap of 4.5 mm has an permanent load of 0.37 kPa, according to Table 2.23 of the Red Book (SAISC, 2013). The Vastrap was assumed to be fixed over a span of 1.6 m in the structures the participant design, with it being welded to the beams to ensure a fixed connection. This changed the minimum and maximum loads which were specified for industrial structures to the values shown in Table 7.15 for light industrial structures. The adopted load weighting factors for Vastrap are demonstrated by the orange line on Figure 7.10.

Concrete floors on metal deck with a 140 mm thickness and spans ranging from 2.5 m to 2.8 m are normally used in light industrial structures. The permanent load of these floors range between 2.52 kPa and 2.56 kPa, according to p.14.24 to p.14.25 of the Red Book (SAISC, 2013).

The popular distributed loads are shown in Table 7.13. These loads are a combination of the popular spans with the factored permanent and imposed loads. The maximum and minimum loads were the same as for industrial structures. The load weighting factors for concrete floors on metal deck are demonstrated by the green line in Figure 7.10.

Table 7.15: Range of ultimate distributed loads in light industrial structures

Floor systems	**Minimum load (kN/m)**	**Popular load range (kN/m)**	**Maximum load (kN/m)**	**Span range considered (m)**
Grating	4.0	12.6	24.0	1.5
Vastrap	4.8	10.1	19.2	1.6
Concrete on metal deck	6.0	27.7 - 31.0	46.6	2.0 - 2.8

The same method was used for light industrial structures as for industrial structures in order to combine all the weighting factors into one combined weighting factor curve, as represented by the red line in Figure 7.10. The popularity of the light industrial floor systems is provided in Table 7.14.

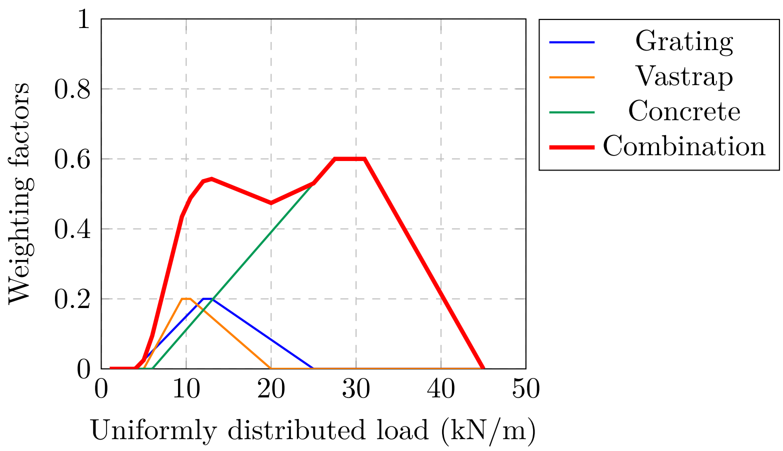


Figure 7.10: Load weighting factors for light industrial structures of Weighting B (f_{load})

Office blocks

Participants 2 and 4 have the most experience in office blocks and their contributions were therefore considered in the development of the load weighting factors for office blocks (refer to Section 7.3.3.3).

The imposed loads normally range between 1.5 kPa to 3 kPa in office blocks and the permanent loads in these structures normally include the own weight of one of the following floor systems: in-situ concrete floors, hollow core concrete floors or concrete on metal deck floors. The popularities of these floor systems are shown in Table 7.16.

Table 7.16: Popularity of office floor systems

Floor systems	**Popularity (%)**
In-situ concrete	80.8
Hollow core	11.5
Concrete on metal deck	7.7

The **in-situ concrete floors** are normally constructed with permanent shuttering on I-sections. The permanent load of the shuttering was not included in the calculations, as there are several different permanent shuttering systems on the market, each with a different own weight. The own weight of shuttering is always small compared to the weight of the concrete floor and will thus not make a big difference in the end result.

In-situ concrete floors ranging from 140 to 170 mm with spans of 2 m are normally used in office blocks, according to the participants. The permanent load of these floors range between 3.43 kPa to 4.17 kPa when a concrete unit weight of 24.5 kN/m^3 (2 % reinforced concrete) is assumed, according to Robberts and Marshall (2010). An additional permanent load of 1.13 kPa was also added to account for a 50 mm screed layer on the concrete floors, assuming a unit weight of 22.6 kN/m^3 (plain unreinforced concrete) for screed (Robberts and Marshall, 2010). The combination of the spacing with the factored imposed and permanent loads yielded the popular distributed load, as shown in Table 7.17.

The maximum and minimum distributed loads of in-situ concrete are shown in Table 7.17. The minimum distributed load corresponds to the factored own weight of a 140 mm thick slab (without imposed loads) and maximum distributed load to the factored load of a 255 mm thick slab (including the imposed loads), both at a span of 2 m. The adopted load weighting factors for in-situ concrete floors are demonstrated by the blue line in Figure 7.11.

Hollow core concrete floors typically range from 150 to 200 mm in office blocks, with permanent loads ranging from 2.6 kPa to 3.1 kPa, according to Topfloor's website (TOPFLOOR, 2016b). The popular factored distributed loads of hollow core concrete floors are shown in Table 7.17 and were calculated with a typical span of 2 m. A permanent load of 1.13 kPa was also added to account for a 50 mm screed layer.

The maximum and minimum distributed loads shown in Table 7.17 correspond to the factored own weight of a 150 mm thick hollow core slab (without imposed loads) and the capacity of a 250 mm thick slab (own weight and imposed loads included), both at a span of 2 m. The adopted load weighting factors for hollow core concrete floors are demonstrated by the orange line in Figure 7.11.

Participants 2 and 4 normally use **concrete floors on metal deck** ranging from 140 to 170 mm in office buildings, with permanent loads ranging from 2.52 kPa to 3.27 kPa, according to p.14.24 to p.14.25 of the Red Book (SAISC, 2013). These floors normally span 2 m and correspond to the factored popular loads presented in Table 7.17. The maximum and minimum factored loads for office blocks are the same as for industrial structures. The load weighting factors for concrete floors on metal deck are demonstrated by the green line in Figure 7.11.

Table 7.17: Range of distributed loads in office blocks

Floor systems	**Minimum load (kN/m)**	**Popular load range (kN/m)**	**Maximum load (kN/m)**	**Span range considered (m)**
In-situ concrete	8.3	15.7 - 22.3	27.3	2.0
Hollow core	8.9	13.7 - 19.8	43.4	2.0
Concrete on metal deck	6.0	13.6 - 20.2	46.6	2.0

The same method was used for office blocks as for the other structures in order to combine all the weighting factors into one combined weighting factor range, as presented in Figure 7.11. The popularities of the various office floor systems are presented in Table 7.16.

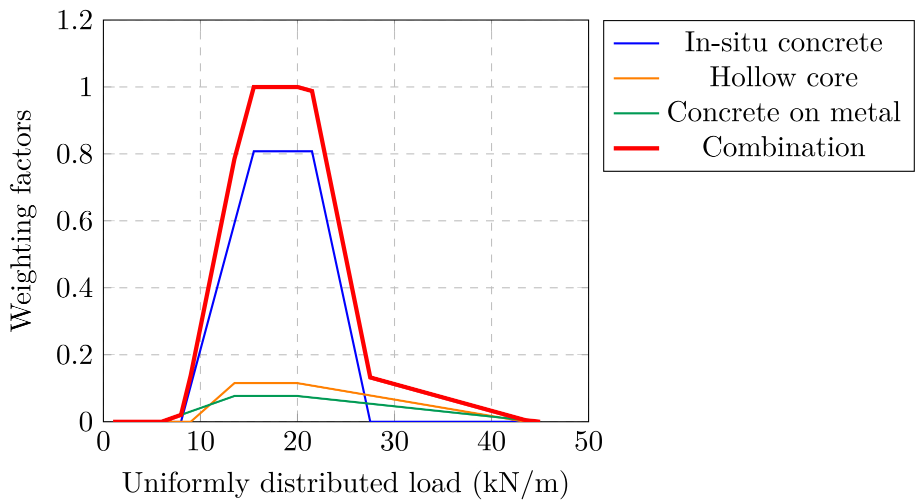


Figure 7.11: Load weighting factors for office blocks of Weighting B (f_{load})

Combination of all structures

The load weighting factor graphs of the three types of structure were combined into one load weighting factor graph, representing all the types of structure mentioned. The graphs were combined according to the frequency of occurrence relative to all the structures. Of the structures, 80.8 % were considered to be industrial structures, 6 % light industrial structures and 13.1 % office blocks (refer to Table D.3 of Appendix D). The final combined weighting factor graph is presented in Figure 7.12.

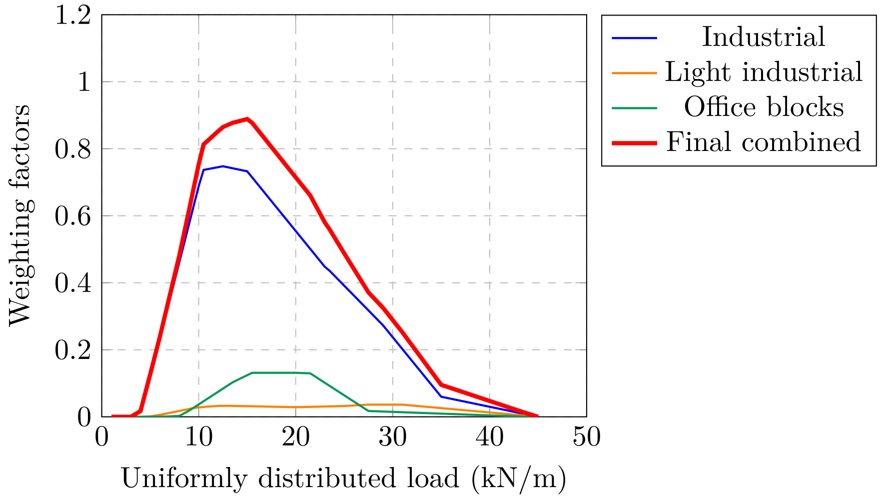


Figure 7.12: Final load weighting factors for all structures of Weighting B (f_{load})

7.4.3 Comparison of weighting factors of Weighting A and B

7.4.3.1 Weighting factors for lateral support conditions

Weighting A regarded laterally supported and unsupported conditions as equally popular, thus effectively disregarding lateral support conditions.

Weighting B did define weighting factors for different lateral support conditions, with laterally unsupported conditions being regarded as more popular than laterally supported conditions (refer to Section 7.4.2.1).

7.4.3.2 Weighting factors for span

As shown on Figure 7.13, Weighting A regarded spans of 4 m to 8 m to be the most common beam spans, dropping off to spans of 2 m and 10 m, while Weighting B regarded spans of 2 m to 3 m as the most common beam spans; much shorter than with Weighting A. Although the weighting factors for Weighting A and B differ to a large extent, they still considered more or less the same span ranges.

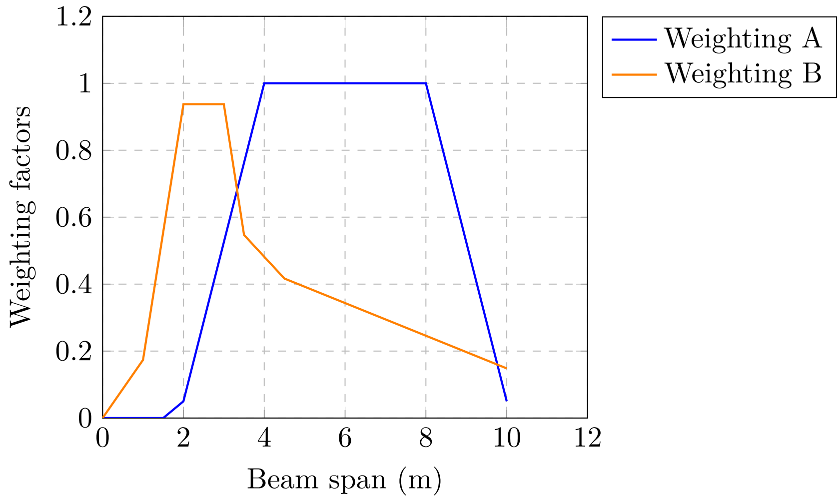


Figure 7.13: Comparison of span weighting factors (f_{span}) of Weighting A and B

7.4.3.3 Weighting factors for loading

Weighting A regarded a larger part of the design space as popular compared to Weighting B. Weighting A regarded uniformly distributed loads of 18 kN/m to 27 kN/m as the most popular loads, while Weighting B regarded 12 kN/m to 16 kN/m as the more popular loads. The load weighting factors of Weighting A and B also differed considerably, but were much closer to each other than the span weighting factors, considering more or less the same load ranges.

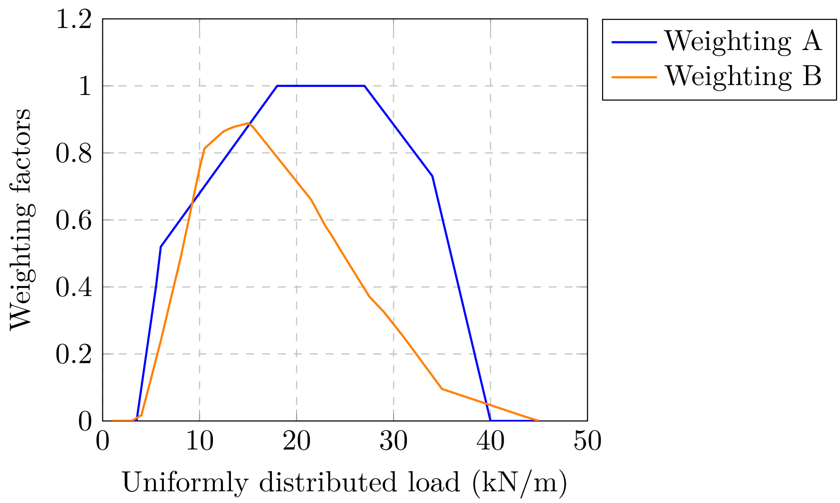


Figure 7.14: Comparison of load weighting factors (f_{load}) of Weighting A and B

7.4.3.4 Final comparison of Weighting A and B

Although Weighting A and B differ significantly, both are valid representations of the popularity of design parameters in practice.

Weighting B should provide a better representation of popular design parameters in practice, because it is based on real information from industry. Nevertheless, Weighting A was still used to determine how sensitive the optimal set of welded I-sections is to the distribution of the popularity weighting factors.

7.5 Popularity weighting factors for GIRDER conditions (Weighting C)

The weighting factors in this section account for different girder design parameters, which are defined according to girder span, point load, load spacing (beam spacing), effective length and lateral support conditions. Per definition, girders support beams (point loads) and have effective lengths shorter than their span, depending on where the compression flange is supported.

This section does not include a weighting corresponding to the "Preliminary weighting factors", as it was found that "Weighting factors based on the survey" yielded a better representation of the popular design parameters in practice (refer to Section 7.4.3.4).

Weighting C, the "Weighting factors for girders based on survey" specifies popularity weighting factors for Design Space C. It specifies weighting factors for 18060 different girder conditions (refer to Section 5.3.2), ranging from 1 m to 10 m spans, with 0.5 m to 6 m effective lengths and 10 kN to 350 kN point loads, at spacing between 0.5 m to 4.5 m, including different lateral support conditions.

Only the information from Participants 1 and 2 was used to determine the weighting factors presented in this section. Participants 3 and 4 do not make use of girders in their structures (refer to Appendix C).

7.5.1 Weighting factors for lateral support conditions

The same process was followed as presented in Section 7.4.2.1 to obtain the popularity of laterally supported and unsupported conditions for girders. The popularity of the laterally supported conditions was found to be 26.7 % and 73.3 % for laterally unsupported conditions (refer to Table D.4 of Appendix D), with lateral support weighting factors ($f_{support}$) of 0.27 and 0.73 respectively.

"Laterally supported" means that the section is continuously laterally supported over the span of the section and that the effective length of the section is equal to zero.

7.5.2 Weighting factors for span

The span weighting factors (f_{span}) of Weighting C were determined with the same process as for Weighting B, but just with girder spans. The number of structures considered to obtain the span weighting factors of Weighting C is presented in Table 7.18, with the calculations in Appendix D. The weighting factors for each structure were combined to form the "Combined factors", as presented in Figure 7.15. These weighting factors are represented by the "Calculated" series line in Figure 7.16.

Table 7.18: Number of structures considered for span weighting factors of Weighting C

Structures	**Participants**	**Number of structures**	**Frequency of occurrence (%)**
Industrial	1	50	55.56
	2	30	33.33
Office	2	10	11.11

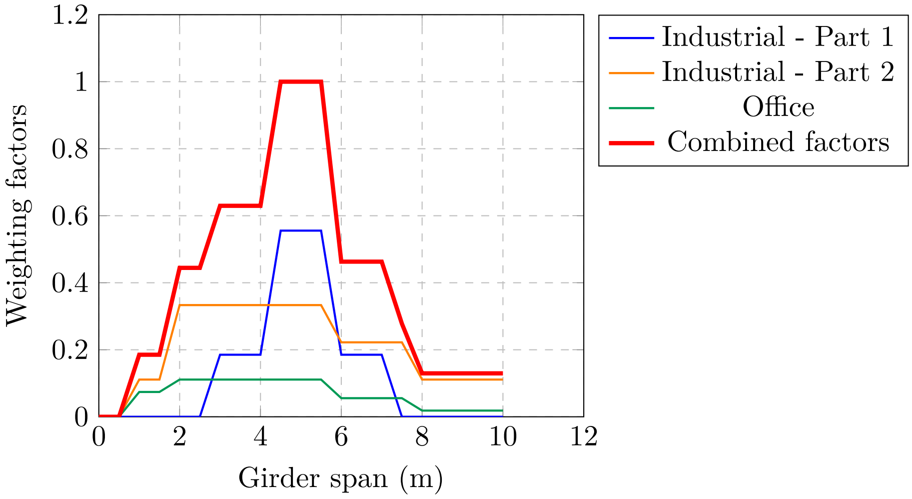


Figure 7.15: Girder span weighting factors of Weighting C (f_{span})

Spans of between 4.5 m and 5 m were found to be the most popular, dropping off on both sides to a minimum span of 1 m and a maximum span of 10 m. Refer to Appendix D for the calculations. The "Calculated" curve was subsequently smoothed out to yield the "Practical" red line in Figure 7.16, for use in the optimisation process.

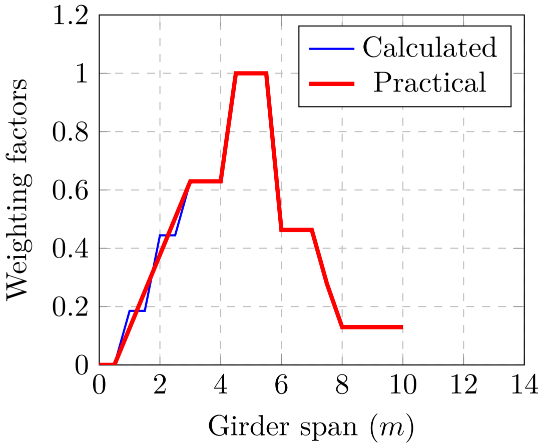


Figure 7.16: The final girder span weighting factors of Weighting C (f_{span})

7.5.3 Weighting factors for effective length

The weighting factors for effective length ($f_{effective}$) were determined from the information presented in Section 7.3.7 and Table 7.1. Participant 1 uses effective lengths of 3 m (60 % of the time) and 5 m (40 % of the time), while the effective lengths of Participant 2 correspond to a beam spacing of 1.5 m and 2 m. With the use of the popularity of the floor systems of Participant 2, it was found that 1.5 m spacing occurs about 52.5 % of the time and a 2 m spacing 47.5 % of the time.

The mentioned popularity percentages were factored accordingly, to obtain weighting factor values of 1.0 for the corresponding popular effective lengths. The most popular effective length of Participant 1 was 3 m and for Participant 2 it was 1.5 m. The rest of the weighting factors of Participant 1 dropped off to 0.5 m and 6 m, while the weighting factors of Participant 2 dropped off to 0.5 m and 3 m (shown in Figure 7.17).

The weighting factors from Participants 1 and 2 were combined in proportion to the number of structures they design per year in order to determine the "Combined" weighting factors presented in Figure 7.17. Participant 1 designs approximately 50 structures per year, which represents 55.56 % of the structures considered. Participant 2 designs approximately 40 structures per year, which represents 44.44 % of the structures considered. The "Combined" weighting factors in Figure 7.17 were used in the optimisation process for girder conditions.

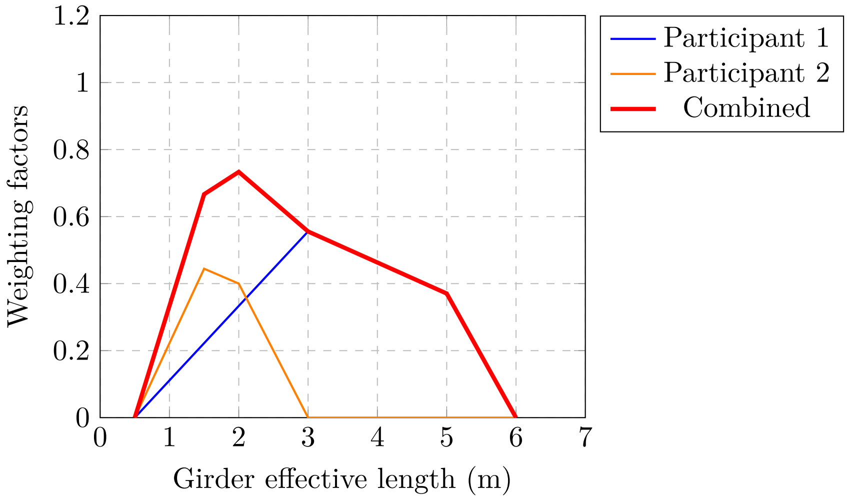


Figure 7.17: Effective length weighting factors of Weighting C ($f_{effective}$)

7.5.4 Weighting factors for loading

The weighting factors in this section were determined from the information presented in Sections 7.4.2.3 and Table D.2 of Appendix D. All the loads in this section are factored loads, also referred to as ultimate loads.

The range of uniformly distributed loads from Sections 7.4.2.3 was multiplied by the popular spans of each structure to determine the popular point loads. The permanent loads of the commonly used I-sections from Section 7.3.1.1 were then added to the point loads. Table 7.19 presents these sections with their corresponding own weights obtained from the Red Book (SAISC, 2013), which were added to the calculated point loads.

Table 7.19: Typical I-beams used in practice

	Section	**m (kg/m)**	**Ultimate permanent load (kN/m)**
Smallest I-beam	IPE_{AA} 160	12.3	0.14
Smallest popular I-beam	IPE 200	22.4	0.26
Largest popular I-beam	254 x 146 x 31	31.1	0.37
Largest I-beam	457 x 191 x 67	67.1	0.79

The weighting factors of two structures were firstly considered separately, namely industrial structures and office buildings, after which the weighting factors of the structures were combined according to the number of structures considered for each participant, in order to obtain final weighting factors.

Industrial structures

Beam spans of 2 to 3 m were found to be the most popular in industrial structures (refer to Appendix D). The popular distributed loads shown in Table 7.13 plus the extra beam loads from Table 7.19 were multiplied by the beam spans to obtain the popular point loads shown in Table 7.20. The minimum and maximum distributed loads were multiplied by beam spans of 1.5 m and 10 m respectively, with the minimum distributed load of the concrete on metal deck being the only exception. The minimum load of the concrete on metal deck was multiplied by a beam span of 2 m.

Table 7.20: Range of ultimate point loads in industrial structures

Floor systems	**Minimum load (kN)**	**Popular load range (kN)**	**Maximum load (kN)**
Grating	6.2	21.6 - 46.1	358.4
Vastrap	5.2	20.8 - 37.8	238.2
Concrete on metal deck	12.4	48.0 - 87.3	474.1

Each range of weighting factors was factored according to the corresponding popularity of each floor system presented in Table 7.12 and then added together to form the combined weighting factor range for industrial structures, as presented in Figure 7.18.

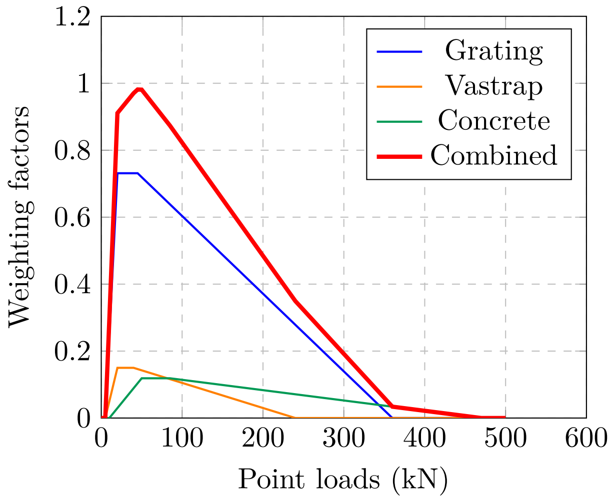


Figure 7.18: Load weighting factors for industrial structures of Weighting C (f_{load})

Office blocks

Beam spans of 2 m to 5.5 m were found to be the most popular in office blocks (refer to Appendix D). The popular distributed loads shown in Table 7.17 plus the extra beam loads from Table 7.19 were therefore multiplied by these beam spans in order to obtain the popular point loads shown in Table 7.20. The minimum and maximum distributed loads were multiplied by beam spans of 2 m and 10 m respectively.

Table 7.21: Range of ultimate point loads in office blocks

Floor systems	**Minimum load (kN)**	**Popular load range (kN)**	**Maximum load (kN)**
In-situ concrete	16.8	32.0 - 124.7	281.0
Hollow core	18.1	27.9 - 110.9	442.4
Concrete on metal deck	12.4	27.6 - 112.9	474.1

The same method was used for office blocks as for industrial structures in order to combine all the weighting factors into one combined weighting factor curve, as presented in Figure 7.19. The popularities of the various office floor systems are presented in Table 7.16.

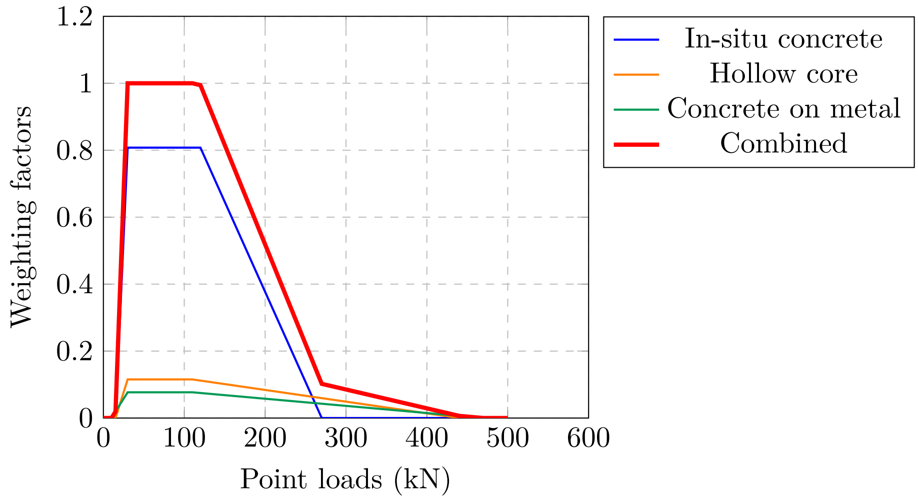


Figure 7.19: Load weighting factors for office blocks of Weighting C (f_{load})

Combination of all structures

All of the load weighting factors mentioned was combined into one load weighting factor graph, representing all of the structures mentioned. Graphs were combined according to the frequency of occurrence of structures, with 86 % of the structures considered to be industrial structures and 14 % office blocks (refer to Table D.6 of Appendix D). The final combined weighting factor graph is presented in Figure 7.20.

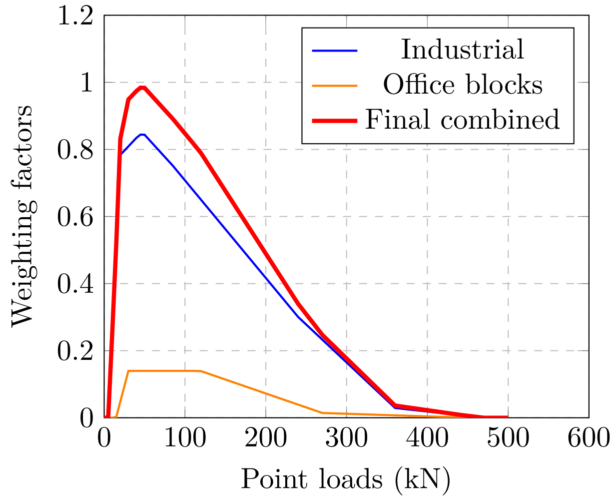


Figure 7.20: Final load weighting factors for all structures of Weighting C (f_{load})

7.5.5 Weighting factors for load spacing

The range of beam spacing used to obtain the distributed loads in Section 7.4.2.3 was combined to obtain weighting factors for load spacing ($f_{spacing}$), as presented in this section.

Industrial structures

The beam spacing in industrial structures range from 0.5 m to 4.5 m, depending on the floor system used. Table 7.22 provides typically used beam spacing, with the maximum and minimum spacing used for each of the three floor systems used in industrial structures.

Table 7.22: Beam spacing used in industrial structures

Floor systems	**Minimum spacing (m)**	**Typical spacings (m)**	**Maximum spacing (m)**
Grating	0.5	1.25 - 1.5	2.5
Vastrap	0.6	1.2 - 1.4	2
Concrete on metal deck	2	2 - 2.5	4.5

The typically used (popular) beam spacing was allocated a weighting factor of 1.0 and the weighting factor graph then dropped off to the minimum and maximum spacing. The weighting factors for each floor system were then combined according to its popularity, presented in Table 7.12, in order to formulate the combined weighting factor graph, as shown in Figure 7.21.

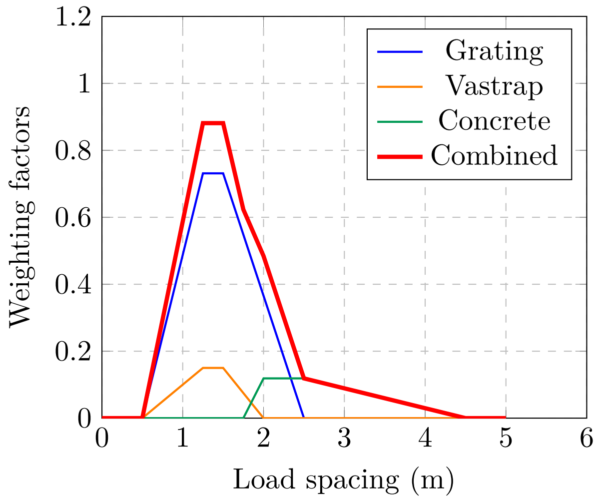


Figure 7.21: Load spacing weighting factors for industrial structures of Weighting C ($f_{spacing}$)

Office blocks

The beam spacing in office blocks ranges from 2 m to 4.5 m depending on the floor system used. Table 7.23 provides the typically used beam spacing, with the maximum and minimum spacing used for each of the three floor systems used in office blocks.

Table 7.23: Beam spacing used in office buildings

Floor systems	Minimum spacing (m)	Typical spacings (m)	Maximum spacing (m)
In-situ concrete	2	2	2
Hollow core	2	2 - 2.5	4.5
Concrete on metal deck	2	2 - 2.5	4.5

The same method was used for office blocks as for industrial structures in order to combine all the weighting factors into one combined weighting factor range, as presented in Figure 7.22. The popularities of office floor systems are presented in Table 7.16.

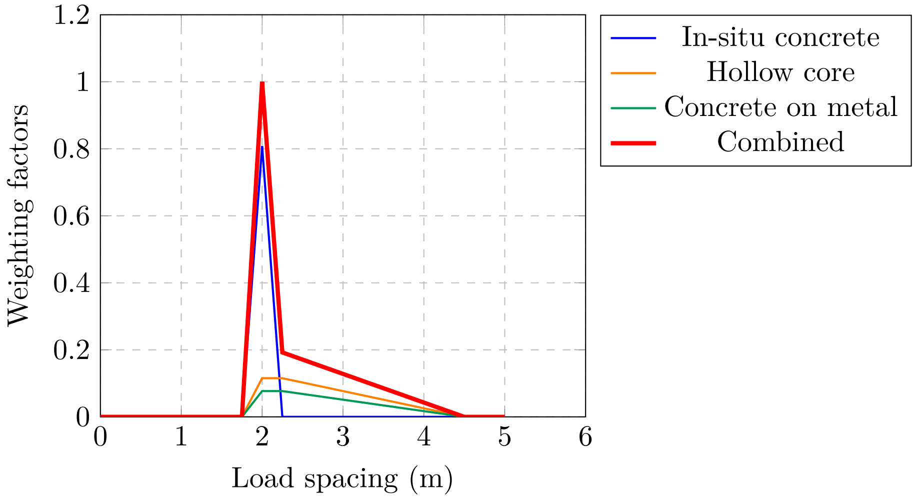


Figure 7.22: Load spacing weighting factors for office blocks of Weighting C ($f_{spacing}$)

Combination of all structures

All of the load weighting factors mentioned was combined into one load weighting factor graph which represents all the structures mentioned. The weighting factors were combined according to the frequency of occurrence of the structures, which were the same for the weighting factors for loading and weighting factors for load spacing (refer to Section 7.5.4). The final combined weighting factor graph is presented in Figure 7.23.

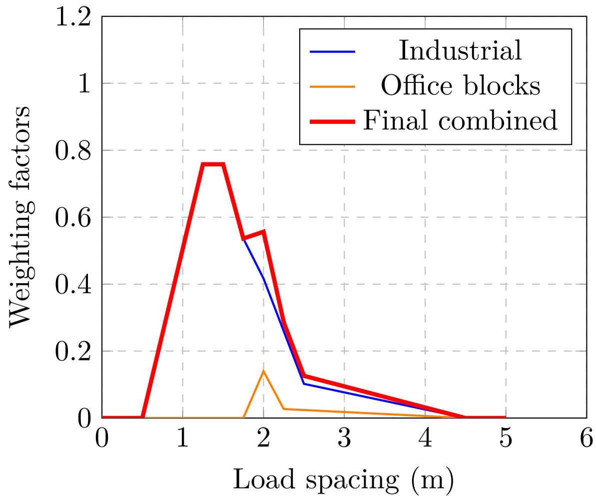


Figure 7.23: Load spacing weighting factors for all structures of Weighting C ($f_{spacing}$)

7.6 Conclusion

This chapter provides three different sets of popularity weighting factors, Weighting A, B and C. All three sets were used in the optimisation methodology, presented in Chapter 5, in order to determine the different optimal sets of welded I-sections for the different design parameter ranges.

The design spaces and assumptions of the participant engineers do not necessarily represent the entire population of engineers of South Africa, as they were limited to four engineers. Nevertheless, their information were used without question to produce the popularity weighting factors. The popular design parameters from practice were thus accepted instead of using the design parameters which are thought to be popular, which gave more power to the approach.

Chapter 8

Discussion of results

8.1 Introduction

A number of different optimal sets were obtained based on the methodology presented in Chapter 5. These optimal sets were obtained, starting with different initial sets (presented in Chapter 4), using different popularity weighting factors (presented in Chapter 7).

The obtained optimal sets are discussed in this chapter and can be broken up into two different groups, namely I-sections for beam conditions and I-sections for girder conditions. The design parameters of these two groups were defined differently, as described in Chapter 5.

8.2 Optimal sets for beam conditions

Three initial sets, Initial Sets 1, 2 and 3 (Sections 4.7 and 4.7) and two sets of popularity weighting factors, Weighting A and B (refer to Section 7.4), were used to obtain the different optimal sets for beam conditions. Weighting A defined weighting factors for 2686 different data points and Weighting B for 3560 data points.

Initial Set 1 was created by approximating all the hot-rolled I-sections available globally, while Initial Sets 2 and 3 were created based on practical considerations and different increments in the dimensions of constituent plates. Table 8.1 provides more information on these initial sets.

Table 8.1: Initial sets used for beam conditions

Sets	**Web increment size (mm)**	**Flange increment size (mm)**	**Initial number of sections**	**Number of practical sections**
Initial Set 1	N/A	N/A	692	608
Initial Set 2	10	10	2,394,896	2,394,896
Initial Set 3	50	20	255,663	255,663

The minimum rating and allowable overlap percentages used in the optimisation methodology (see discussion in Sections 5.4 and 5.7) were varied to obtain the most optimal set of welded I-sections. The minimum rating was varied between 75 and 100 in Step 3 of the methodology. The allowable overlap percentage was gradually reduced by 2.5 % or 5 %, from 100 % to a minimum of 70 % until the Initial Optimal Set contained between 70 and 80 sections in Step 6 of the methodology for beams (refer to Section 5.7) and approximately 150 sections for girders.

The Initial Optimal Set was then reduced in Step 7 to the Final Optimal Set. The Final Optimal Set contained all the lightest sections of each data point from the Initial Optimal Set (refer to Section 5.8).

Each optimal set in this chapter is identified by its set of popularity weighting factors, followed by its initial set. For example, an optimal set created with the popularity weighting factors of Weighting A and based on Initial Set 1 is defined as an A1 Optimal Set. With this information the optimal sets can also be identified more precisely with its minimum rating, which was used in Step 3 of the optimisation process.

8.2.1 Optimal sets from Initial Set 1

Table 8.2 presents the number of sections of the A1 and B1 Optimal Sets at different stages of the optimisation process: after Step 5, during Step 6 and at the end of the optimisation process (final).

All the A1 and B1 Optimal Sets were obtained with an allowable overlap percentage of 100 %, excluding the optimal sets obtained with a minimum rating of 75 and 80. These optimal sets had to be reduced with an overlap percentage less than 100 % in order to obtain an optimal set that meets the required size of 70 to 80 sections, in Step 6.

Table 8.2: Number of sections in optimal sets A1 and B1

	Number of sections in set					
Minimum rating of set	**After Step 5**		**Step 6 (start to end)**		**After Step 7 (final)**	
	Weighting A	**Weighting B**	**Weighting A**	**Weighting B**	**Weighting A**	**Weighting B**
75	278	282	81 - 73	82 - 72	55	55
80	244	244	79	81 - 69	61	58
85	200	205	73	74	57	59
90	162	164	62	62	53	56
95	132	132	62	64	56	57
100	83	87	82	85	82	85

The number of sections in the optimal sets obtained from Initial Set 1 was not sensitive to the popularity weighting factors (Weighting A or B), as shown in Table 8.2. The size of the A1 and B1 Optimal sets differed slightly from each other after Step 5, because of the higher number of data points in Design Space B.

The number of sections in the optimal sets was however sensitive to the variation in minimum rating, as the number of sections,, after Step 5, varied a lot with the incremental increase in minimum rating (refer to Table 8.2).

Figure 8.1 and 8.2 show how the I-sections were reduced from Step 5 to Step 7. The optimal sets obtained with a minimum rating of 100 were however not reduced from Step 6 to 7, as it already contained the lightest sections possible with Initial Set 1, as shown in the figures below.

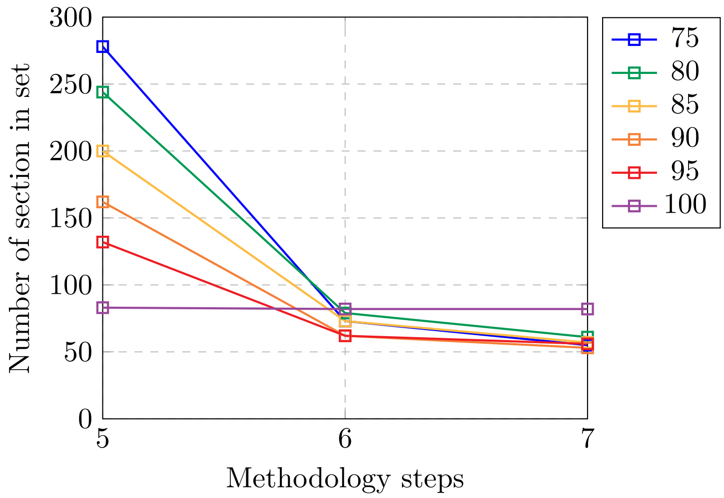


Figure 8.1: Number of sections from Step 5 to Step 7 (A1)

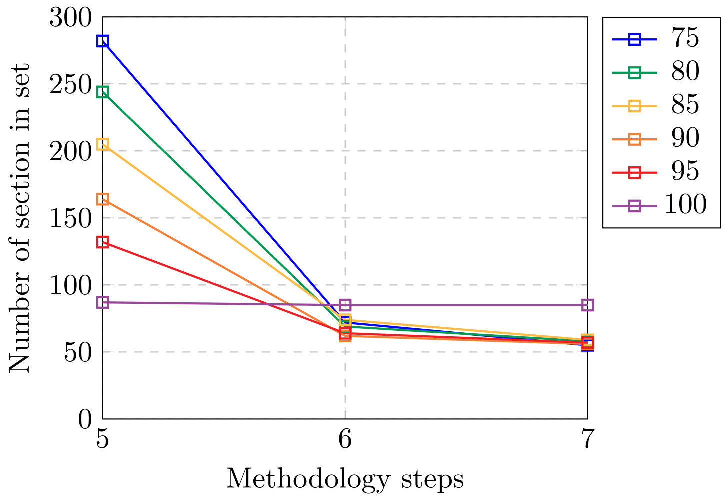


Figure 8.2: Number of sections from Step 5 to Step 7 (B1)

The Final A1 and B1 Optimal sets were close to each other in size, as shown in Table 8.2. This was however not the case with the other optimal sets, as shown in Table 8.6 and 8.9. For this reason it was decided to compare the Initial Optimal Sets (from Step 6) with each other, rather than the Final Optimal Sets, to determine the sensitivity of the optimisation process to the different parameters. The Initial Optimal Sets had to be reduced to a required size (70 to 80 sections), making it easier to compare with each other.

Table 8.3 and 8.4 show the extent to which the Initial Optimal Sets correspond to each other. The table values represent the percentage of all the individual sections in an optimal set based on one minimum rating (in left column), that can also be found in an optimal set that is based on another minimum rating value. The blue highlighted column corresponds to the lightest optimal set possible, based on Initial Set 1, obtained with a minimum rating of 100.

Table 8.3: Percentage of same sections in Initial A1 Optimal Sets (%), depending on minimum rating

Minimum rating of first set	**Minimum rating of second set**					
	75	**80**	**85**	**90**	**95**	**100**
75	100.00	84.93	75.34	67.12	63.01	68.49
80		100.00	75.95	67.09	63.29	73.42
85			100.00	72.60	68.49	75.34
90				100.00	72.58	80.65
95					100.00	90.32
100						100.00

Table 8.4: Percentage of same sections in Initial B1 Optimal Sets (%), depending on minimum rating

Minimum rating of first set	**Minimum rating of second set**					
	75	**80**	**85**	**90**	**95**	**100**
75	100.00	80.56	75.00	66.67	62.50	70.83
80		100.00	75.36	69.57	62.32	75.36
85			100.00	68.92	70.27	77.03
90				100.00	74.19	82.26
95					100.00	89.06
100						100.00

The A1 and B1 Initial Optimal Sets corresponded by more than 60 % to each other and more than 68 % to the lightest set (blue highlighted column), regardless of the weighting factors or minimum rating, as shown in Table 8.3 and 8.4. However, the closer the minimum rating was to 100, the closer the optimal set was to the lightest optimal set. Both Initial A1 and B1 Optimal Sets are thus sensitive to the different minimum rating values, notwithstanding the different weighting factors, as the correspondents are more or less the same between Table 8.3 and 8.4.

It was found that all the section from the Final A1 and B1 Optimal Sets can be obtained from a set of 94 welded I-sections. Furthermore, all the A1 and B1 Final Optimal Sets which were based on a minimum rating between 75 and 95 contained 30 of the same sections. These sections differed slightly between the A1 and B1 Optimal Sets. Of the 30 sections, 28 (93 % of the popular sections) were the same for both of the Final Optimal Sets, A1 and B1. The Final Optimal Sets obtained from Initial Set 1 are thus not sensitive to different popularity weighting factors, but are indeed sensitive to different minimum ratings, as the popular I-sections only correspond to approximately 49 % of a Final Optimal Set obtained from Initial Set 1. The popular I-sections from both Final Optimal Sets can be found in Table E.1 of Appendix E.

It was also found that the top three sections of the Final B1 Optimal Sets tended to be smaller compared to the top three sections of the Final A1 Optimal Sets. The reason for this being that Weighting A regarded design problems with larger loads and spans to be more popular than Weighting B. The four sections shown in Table 8.5 were found, most of the time, under the top three sections of the Final A1 and B1 Optimal Sets.

Table 8.5: Top sections from Final A1 and B1 Optimal sets

Section Name $h_w^* \times b \times g$	h_w^* **mm**	b **mm**	t_w **mm**	t_f **mm**
160.0 x 105.0 x 19.47	160	105	5	8
240.0 x 100.0 x 17.27	240	100	5	5
290.0 x 100.0 x 20.8	290	100	5	6
380.0 x 140.0 x 35.48	380	140	6	8

The 240 x 100 x 17.27 welded I-section was the only section that was always under the top three sections of all the Final A1 and B1 Optimal Sets. This section performed well regardless of the popularity weighting factors or minimum rating.

The 290 x 100 x 20.8 and 380 x 140 x 35.48 welded I-sections were most of the time under the top three sections of the Final A1 Optimal Sets and the 160 x 105 x 19.47 welded I-section most of the time under the top three sections of the Final B1 Optimal Sets.

The 160 x 105 x 19.47 and 240 x 100 x 17.27 welded I-sections, respectively, correspond to the popular IPE 180 and UB 254 x 146 x 31 sections used in South Africa (refer to Section 7.3.1.1). This demonstrates that there are better hot-rolled I-sections available globally with flanges that are smaller and larger than the hot-rolled I-sections used in South Africa.

The 290 x 100 x 20.8 and 380 x 140 x 35.48 welded I-sections correspond, respectively; to the popular UB 305 x 102 x 25 and UB 406 x 140 x 39 sections (refer to Section 7.3.1.1). The UB 305 x 102 x 25 is normally popular under laterally supported conditions.

8.2.2 Optimal sets from Initial Set 2

The number of sections in the A2 and B2 Optimal Sets, at different stages of the optimisation process, can be found in Table 8.6.

Table 8.6: Number of sections in optimal sets A2 and B2

Minimum rating of set	Number of sections in set					
	After Step 5		**Step 6 (start to end)**		**After Step 7 (final)**	
	Weighting A	**Weighting B**	**Weighting A**	**Weighting B**	**Weighting A**	**Weighting B**
75	33,161	35,884	895 - 67	1032 - 71	41	47
80	23,728	25,824	797 - 74	935 - 71	41	47
85	15,417	17,023	648 - 71	766 - 75	52	58
90	8,579	9,668	540 - 75	639 - 82	60	72
95	3,373	3,914	506 - 118	585 - 121	118	119
100	569	649	525	601	525	601

The number of sections in the optimal sets obtained from Initial Set 2, as from Initial Set 1, was not really sensitive to the popularity weighting factors. This is clear from the small differences in the number of sections between the optimal sets obtained with Weighting A and Weighting B, after Step 5 (refer to Table 8.6).

The size of the A2 and B2 Optimal Sets, at different stages of the methodology, was however more sensitive to the defined minimum rating than the optimal sets produced from Initial Set 1, as shown in Table 8.6.

Smaller minimum ratings produced larger sets after Step 5, but were then dramatically reduced in Step 6, as shown in Table 8.6. The larger sets produced after Step 5, based on smaller minimum ratings, were also easier to reduce to the required size in Step 6 than the smaller sets, as demonstrated in Figure 8.3 and 8.4.

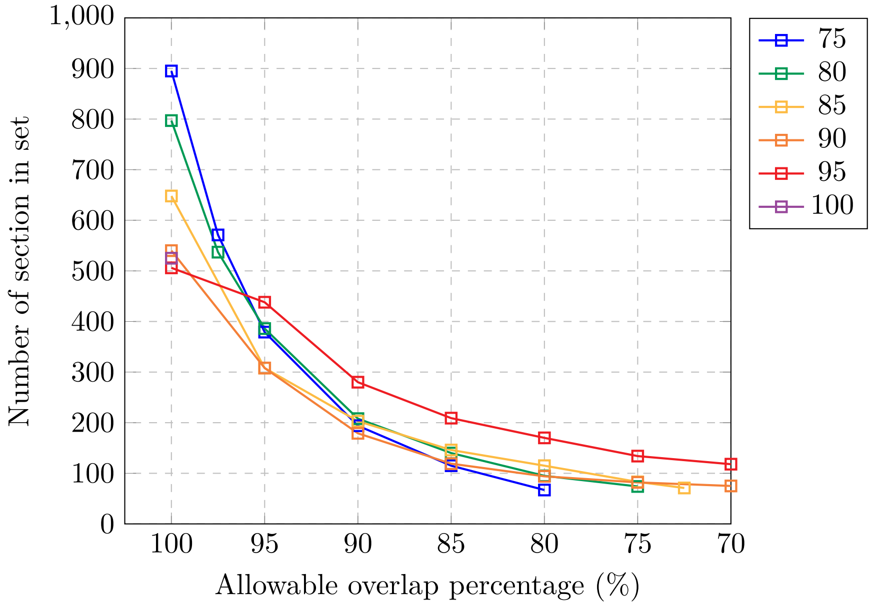


Figure 8.3: Number of sections in the A2 Optimal Sets with different overlap percentages (during Step 6)

Figure 8.3 and 8.4 shows how the size of the optimal set shrunk when the allowable overlap percentage was reduced in the subsequent Step 6 steps of 2.5 % or 5 %. For the A2 Optimal Set with a minimum rating of 75, the overlap percentage had to be reduced from 100 % to 80 % in order to reduce the set to the required size, as shown in Figure 8.3.

When the minimum rating was too high, it was not possible to reduce the optimal set to the required size in Step 6, as shown in Figure 8.3 and 8.4.

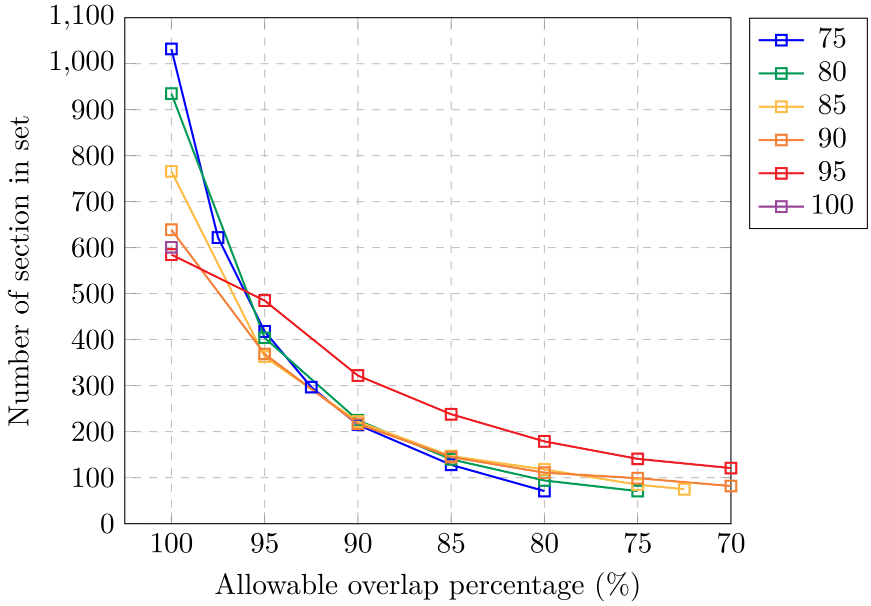


Figure 8.4: Number of sections in the B2 Optimal Sets with different overlap percentages (during Step 6)

Tables 8.7 and 8.8 respectively shows to what extent the A2 and B2 Initial Optimal Sets share the same sections. These sets do not correspond as much as the Initial Optimal Sets obtained from Initial Set 1 (refer to Section 8.2.1). Optimal sets with minimum ratings between 75 and 95, only corresponded between 15 % to 39 % to each other. This is due to the large number of sections (Initial Set 2) to choose from in the process of obtaining an optimal set.

Table 8.7: Percentage of same sections in Initial A2 Optimal Sets (%), depending on minimum rating

Minimum rating of first set	**Minimum rating of second set**					
	75	**80**	**85**	**90**	**95**	**100**
75	100.00	20.90	17.91	17.91	22.39	40.30
80		100.00	21.62	17.57	21.62	39.19
85			100.00	28.17	18.31	59.15
90				100.00	36.00	78.67
95					100.00	97.46
100						100.00

Although the optimal sets were not that similar to each other, as was the case for optimal sets obtained from Initial Set 1, they did compare well with the lightest set, as shown in Table 8.7 and 8.8 (blue column).

Table 8.8: Percentage of same sections in Initial B2 Optimal Sets (%), depending on minimum rating

Minimum rating of first set	**Minimum rating of second set**					
	75	**80**	**85**	**90**	**95**	**100**
75	100.00	21.13	25.35	22.54	15.49	38.03
80		100.00	32.39	18.31	22.54	38.03
85			100.00	24.00	16.00	50.67
90				100.00	39.02	74.39
95					100.00	95.04
100						100.00

It was found that all the sections from the Final A2 and B2 Optimal Sets can be obtained from a set of 559 and 661 welded I-sections respectively, of which 542 sections (between 82 % and 97 %) corresponded between the two groups of optimal sets. This shows that these optimal sets, like the other sets, are not really sensitive to the popularity weighting factors.

Because of the large initial set size, not a single section occurred in all of the Final A2 Optimal sets. Four sections, however, occurred in all of the Final B2 Optimal Sets with a minimum rating between 75 to 95.

Although no single section occurred in all of the Final A2 Optimal sets, 21 sections (± 43 % of each optimal set) occurred in three or more of the five Final A2 Optimal Sets, with a minimum rating ranging between 75 to 95 and 15 sections (± 26 % of each optimal set) in the Final B2 Optimal Sets. These popular I-sections can be found in Table E.4 of Appendix E, of which 9 (± 50 % of the popular I-sections) occurred in both the A2 and B2 Final Optimal Sets.

The Final Optimal Sets produced from Initial Set 2 are thus sensitive to the different popularity weighting factors, the size of the initial set and the variation in minimum rating, as the Final Optimal Sets contained less than four of the same sections in all the different optimal sets. The optimal sets were however more sensitive to the size of the initial set than other parameters, because the correspondents between the optimal set obtained from Initial Set 1 and 3 were higher than those obtained from Initial Set 2 (refer to Sections 8.2.1 and 8.2.3).

As foreseen for the large initial set, there were no sections that constantly appeared under the top three sections in all of the Final Optimal Sets. A difference in section dimensions could however be noticed in the top three most popular sections of the A2 and B2 Final Optimal Sets. The top three sections of the Final A2 Optimal Sets normally consisted of a smaller I-section and two large I-sections, covering the short (lighter) and long (heavier) spans respectively. These spans were regarded as more popular in Weighting A than Weighting B.

The top three most popular sections of the Final B2 Optimal Sets normally consisted of smaller sections that worked for a large number of unpopular design problems, which gave them a high final ranking. The 170 x 100 x 14.52 welded I-section was very popular under the B2 Optimal Sets for this reason (refer to Table E.4 of Appendix E).

8.2.3 Optimal sets from Initial Set 3

The optimal sets obtained from Initial Set 3 behaved similar to the A2 and B2 Optimal Sets under the different popularity weighting factors and minimum ratings. These optimal sets could be reduced easier than the sets obtained from Initial Set 2, as Initial Set 3 contained fewer sections.

Table 8.9 shows the number of sections in the A3 and B3 Optimal Sets at different stages of the optimisation process.

The number of sections of the A3 and B3 Optimal sets after Step 5 also varied with the increase in minimum rating at increments of 5, but varied slightly less than the A2 and B2 Optimal sets.

Table 8.9: Number of sections in optimal sets A3 and B3

	Number of sections in set					
Minimum rating of set	**After Step 5**		**Step 6 (start to end)**		**After Step 7 (final)**	
	Weighting A	**Weighting B**	**Weighting A**	**Weighting B**	**Weighting A**	**Weighting B**
75	3,897	4,250	195 - 69	217 - 76	50	56
80	2,852	3,117	171 - 69	197 - 78	51	61
85	1,952	2,148	157 - 68	181 - 74	56	61
90	1,192	1,326	140 - 78	157 - 74	72	65
95	549	620	128 - 80	144 - 88	79	85
100	184	201	170	187	170	187

Figures 8.5 and 8.6 show how the optimal sets were reduced in size in Step 6, with different allowable overlap percentages, in order to obtain an optimal set with the required size.

The trends of the graphs in Figures 8.5 and 8.6 are different from those in Figures 8.3 and 8.4 in terms of the section reduction rate. The trend difference is due to the number of section possibilities of the initial sets. A set with more section possibilities over the entire design space will initially be reduced at a faster rate in Step 6 than one with fewer possibilities. Initial Set 2 had more initial section possibilities than Initial Set 3, due to the smaller increment sizes used to create the initial set of sections (refer to Table 8.1). The optimal set from Initial Set 3 could therefore be reduced at a faster rate initially. Nevertheless, at the end of Step 6 both reduction rates were similar.

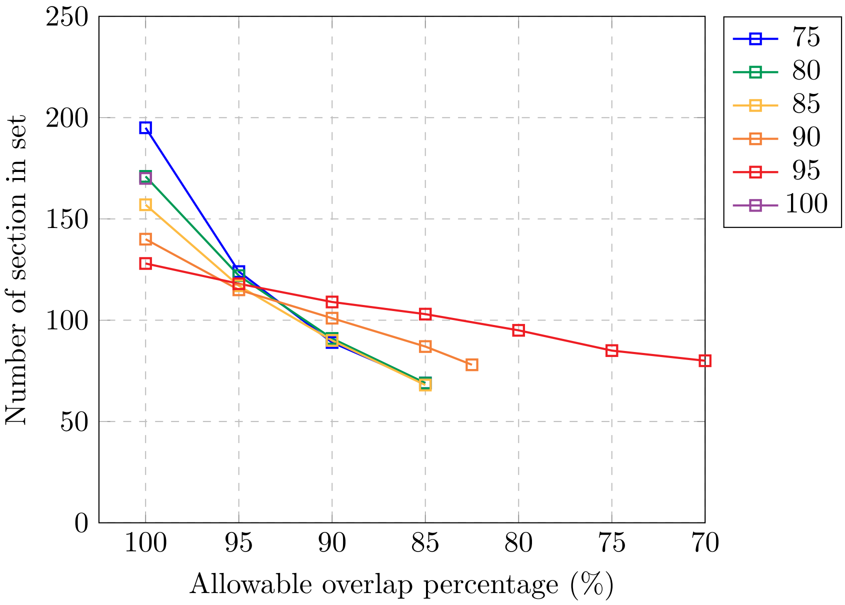


Figure 8.5: Number of sections in the A3 Optimal Sets with different overlap percentages (during Step 6)

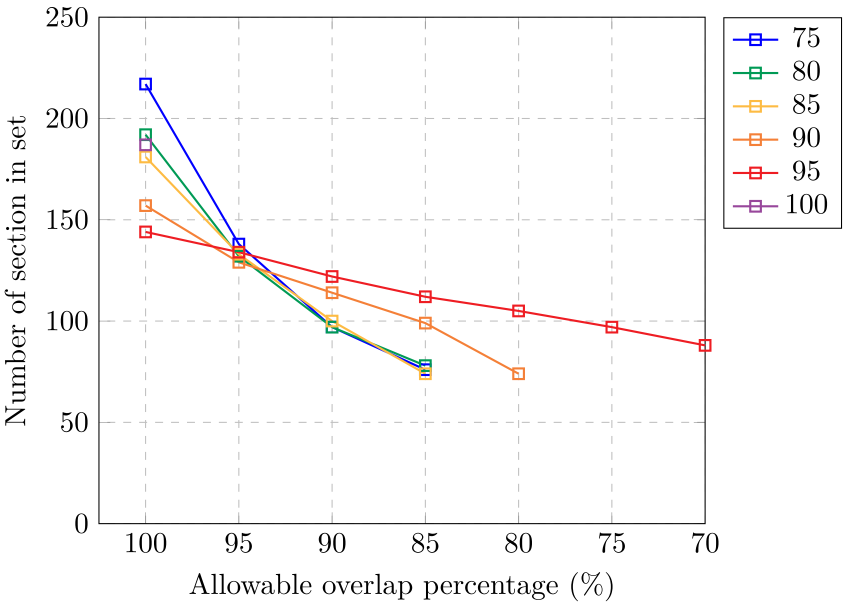


Figure 8.6: Number of sections in the B3 Optimal Sets with different overlap percentages (during Step 6)

Tables 8.10 and 8.11, respectively, show how much the sections in the A3 and B3 Initial Optimal Sets correspond. These Initial Optimal Sets corresponded more to each other than those produced from Initial Set 2 (refer to Section 8.2.2). The Initial Optimal Sets with minimum ratings between 75 and 95 had a correspondence of between 38 % and 70 %. These optimal sets also compared well with respect to the lightest sections, as shown in the tables hereafter.

Table 8.10: Percentage of same sections in Initial A3 Optimal Sets (%), depending on minimum rating

Minimum rating of first set	Minimum rating of second set					
	75	**80**	**85**	**90**	**95**	**100**
75	100.00	69.57	52.17	52.17	40.58	59.42
80		100.00	52.17	55.07	40.58	60.87
85			100.00	61.76	61.76	77.94
90				100.00	70.51	89.74
95					100.00	97.50
100						100.00

Table 8.11: Percentage of same sections in Initial B3 Optimal Sets (%), depending on minimum rating

Minimum rating of first set	Minimum rating of second set					
	75	**80**	**85**	**90**	**95**	**100**
75	100.00	68.42	55.26	48.68	38.16	57.89
80		100.00	55.13	50.00	47.44	62.82
85			100.00	64.86	63.51	72.97
90				100.00	66.22	81.08
95					100.00	96.59
100						100.00

It was found that the Final A3 and B3 Optimal Sets can be obtained from a set of 184 and 207 different welded I-sections respectively, of which 181 I-sections (between 87 % and 98 %) occurred in both the Final A3 and B3 Optimal Sets. This indicates, as for the other optimal sets, that these optimal sets are also not sensitive to the popularity weighting factors.

There were 14 individual sections (± 22 % of each set) that occurred in all the A3 Optimal Sets with minimum ratings between 75 and 95, where 28 sections (± 46 % of each set) occurred in more than four of the five sets. There were also 14 sections (± 22 % of each set) that occurred in all the B3 Optimal Sets with a minimum rating between 75 and 95, where 29 of the sections (± 46 % of each set) occurred in more than four of the sets. Refer to Table E.7 of Appendix E for the dimensions of these popular I-sections.

Of the B3 final optimal sections, 20 (± 70 % of the popular I-sections) corresponded to the A3 sections, which occurred in four of the five optimal sets. The optimal sets produced from Initial Set 3 are therefore not as sensitive to different popularity weighting factors and minimum ratings as the optimal sets obtained from Initial Set 2, with the final optimal sets containing 14 of the same sections in all the different A3 and B3 Optimal Sets, respectively and which is approximately 70 % of the same popular sections.

Unlike the optimal sets obtained from Initial Set 2, the optimal sets obtained from Initial Set 3 had a few sections which were always amongst the top three most popular sections or close to them. Tables 8.12 and 8.13 show these top sections of the A3 and B3 Optimal sets, respectively.

Table 8.12: Top sections from A3 Optimal sets

Section Name $h_w^* \times b \times g$	h_w^* **mm**	b **mm**	t_w **mm**	t_f **mm**
200.0 x 100.0 x 15.7	200	100	5	5
250.0 x 100.0 x 17.66	250	100	5	5
300.0 x 100.0 x 19.62	300	100	5	5
350.0 x 100.0 x 21.59	350	100	5	5

Table 8.13: Top sections from B3 Optimal sets

Section Name $h_w^* \times b \times g$	h_w^* **mm**	b **mm**	t_w **mm**	t_f **mm**
150.0 x 100.0 x 13.74	150	100	5	5
150.0 x 120.0 x 17.19	150	120	5	6
200.0 x 100.0 x 15.7	200	100	5	5

As in the Final A2 Optimal Sets, the top three sections in the Final A3 Optimal Sets tended to be larger than those in the Final B3 Optimal Sets due to the higher popularity weighting factors in the large span and heavy load region of the design space, as shown in Table 8.12.

8.3 Optimal set of girders conditions

Only one initial set (Initial Set 4) and one set of popularity weighting factors (Weighting C) were used to obtain the different optimal sets for girder conditions (refer to Chapter 5), as the sensitivity of the optimisation methodology had already been tested with different popularity weighting factors and initial sets under beam conditions. Weighting C provides weighting factors for 18,060 different data points and Initial Set 4 contained 872,735 practical I-sections (refer to Chapter 4 and 7), which was obtained with the use of a 20 mm flange increment and a 50 mm web increment (refer to Section 4.8).

The minimum rating had to be varied between 75 and 100, as it was found that the optimisation methodology is sensitive to the variation in the minimum rating of Step 3.

The optimal sets for girder conditions were identified in the same manner as the optimal sets for beam conditions (refer to Section 8.2).

Table 8.14 presents the number of sections of the C4 Optimal Sets at different stages of the optimisation process: after Step 5, during Step 6 and at the end of the optimisation process (final).

Table 8.14 does not provide the number of sections for Step 6 or at the end of the optimisation process for the C4 Optimal Sets which were obtained with a minimum rating of 75 and 80, as these sets had too many sections after Step 5. Executing Step 6 for the C4 Optimal Set with a minimum rating of 80 and 75, had to be abandoned due to time constraints, as it would have taken more than four weeks just to solve Step 6 of the C4 Optimal Set with a minimum rating of 80.

Table 8.14: Number of sections in the C4 Optimal Sets

	Number of sections in set		
Minimum rating of set	**After Step 5**	**Step 6 (start to end)**	**After Step 7 (final)**
75	52,557	no number	no number
80	39,772	no number	no number
85	27,723	1885 - 200	140
90	17,669	1607 - 331	254
95	9,648	1177 - 463	403
100	1,778	777	777

The number of sections of the C4 Optimal sets after Step 5 also varied with the increase in minimum rating at increments of 5, as the case with other optimal sets. The C4 Optimal Sets were thus also sensitive to the variation in minimum rating.

Figure 8.7 shows how the C4 Optimal Sets were reduced in size in Step 6, with the different allowable overlap percentages. As shown in the figure, all the optimal sets were reduced with the smallest allowable overlap percentage of 70 %. Only the optimal sets obtained with a minimum rating of 85 were close to the number of I-sections used in South Africa (155 sections), including hot-rolled I-sections and plate girders.

As shown in Figure 8.7, smaller overlap percentage increment sizes had to be used to reduce the C4 Optimal Sets, because the allowable overlap percentage failed the criteria of Step 6 when it was reduced to rapidly (refer to Section 5.7). This reaction was caused by the large Design Space C and Initial Set 4.

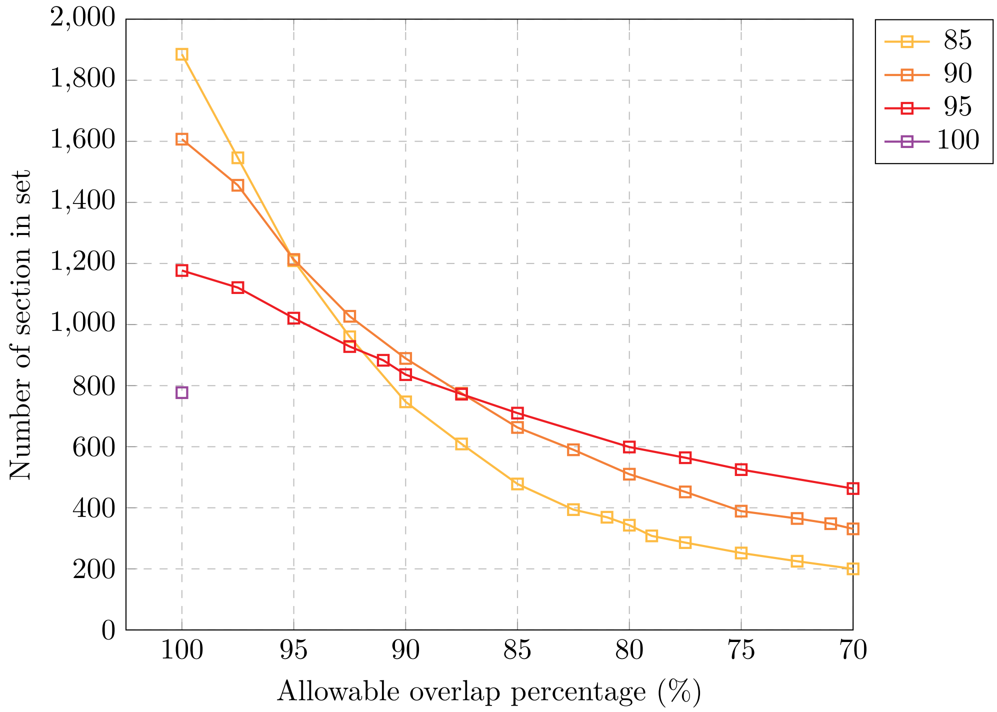


Figure 8.7: Number of sections in the C4 Optimal Sets with different overlap percentages (during Step 6)

The optimal sets obtained from Initial Set 4 could not be reduced to a size of 150 sections in Step 6. For this reason the Final C4 Optimal Sets were rather compared to each other than the Initial C4 Optimal Sets in Table 8.15, as the size of the Initial C4 Optimal Sets could not be reduced to a constant size and thus did not provide any advantage above the Final C4 Optimal Sets.

The Final Optimal Sets in Table 8.15 with a minimum rating between 75 and 95 had a correspondence of between 61 % and 73 %. These optimal sets also compared well with the lightest sections (blue column), as shown in the table below. This shows that the C4 Optimal Sets are sensitive to the variation in minimum rating, but not as sensitive as the optimal sets for beam conditions.

Table 8.15: Percentage of same sections in Final C4 Optimal Sets (%), depending on minimum rating

Minimum rating of first set	**Minimum rating of second set**			
	85	**90**	**95**	**100**
85	100.00	61.43	72.86	85.00
90		100.00	71.65	91.73
95			100.00	89.83
100				100.00

It was found that the Final C4 Optimal Sets consisted of different subsets of a set of 850 different welded I-sections. There were 74 of these sections that occurred in all of the C4 Optimal Sets. This corresponds to 53 % of the Final C4 Optimal Set obtained with a minimum rating of 85. The Final C4 Optimal Sets were thus also sensitive to the variation in minimum rating, but not as sensitive as the final optimal sets obtained from Initial Set 3. These 74 popular I-sections are listed in Table E.11 of Appendix E.

There was no I-section that were constantly present under the top three sections in all of the Final C4 Optimal Sets. The reason for this was that there were too many sections possibilities, within Initial Set 4, and too many design parameters to cover in Design Space C, with 18060 data points, to have sections that are constantly present under the top three sections of all the Final C4 Optimal Sets.

8.4 Conclusion

The sizes of the optimal sets for beam conditions are not really sensitive to different popularity weighting factors, as the number of sections (after Step 5 and at the start of Step 6) in the optimal sets, obtained with Weighting A and Weighting B, do not differ by more than 16 % from each other. This difference also decreases as the initial set decreases.

Although the sizes of the optimal sets for beam conditions are not really sensitive to the different popularity weighting factors, these factors do have a big influence on the ranking of the sections in the Final Optimal Sets, as shown in Table 8.12 and 8.13.

The optimal sets for beam conditions are sensitive to different minimum ratings, as the percentage of shared sections of the initial optimal sets can range from 15 % up to 97 % , as shown in Table 8.7 and 8.8. The size of the initial set does however have a bigger influence on the optimal set comparison than the minimum rating has, as the optimal set comparison ranged between 15 % and 97 % for Initial Set 2 and between 38 % and 97 % for Initial Set 3, as shown in Tables 8.7 to 8.11.

The optimal sets for girder conditions are also sensitive to different minimum ratings, as the percentage of shared sections of the final optimal sets can range from 61 % up to 90 %, as shown in Table 8.15.

The methodology presented in Chapter 5 is therefore regarded as being less sensitive to the popularity weighting factors than to the size of the initial set and the variation in minimum rating.

Chapter 9

Comparison of optimal sets

9.1 Introduction

This chapter compares all the final optimal sets, discussed in Chapter 8, to each other with the use of the comparison methodology presented in Section 9.2 below. This methodology makes use of a virtual "bill of quantities" to compare the optimal sets to each other, after which the most optimal set of welded I-sections is chosen from the different groups of optimal sets, derived from each initial set. These most optimal sets were then compared to each other, including other sets available globally, in order to obtain the best optimal set of welded I-sections.

9.2 Comparison methodology

The methodology used to compare the final optimal sets to each other followed two steps.

The first step obtained, for each set, a single section (the lightest section) corresponding to each data point in the design space. These sections were obtained from Step 7 of the optimisation methodology (refer in Chapter 5).

The second step obtained the total mass (M) of a virtual project using the set of sections from the first step. The virtual project aims to correspond to the average of all the structures built in South Africa in a one year period.

The total mass (M) was obtained by firstly calculating the mass of the member (kg) at each data point, multiplying this mass by the relevant weighting factors (which indicate how frequently that combination of span, lateral support conditions and loading occurs in the project) and adding up the masses. This process is defined in Equation 9.1. Alternatively, an unfactored total mass ($\bar{M}$) can be calculated with Equation 9.2.

$$M = \sum_{j=1}^{n} m_j \cdot L_j \cdot (f_{support.j} \cdot f_{span.j} \cdot f_{effective.j} \cdot f_{load.j} \cdot f_{spacing.j}) \tag{9.1}$$

$$\bar{M} = \sum_{j=1}^{n} m_j \cdot L_j \tag{9.2}$$

where n is the number of data points, m_j is the mass of the (lightest) section at data point j [kg/m], L_j is the section span at data point j [m], $f_{support.j}$ is the lateral support weighting factor, $f_{span.j}$ is the span weighting factor, $f_{effective.j}$ is the effective length weighting factor, $f_{load.j}$ is the load weighting factor and $f_{spacing.j}$ is the load spacing weighting factor. All of the weighting factors are for data point j.

M and $\bar{M}$ will have different values, because all the weighting factors are smaller than or equal to 1. $\bar{M}$ will thus always be larger than M. In calculating $\bar{M}$ all the data points are regarded as equally important.

9.3 Comparison of final optimal sets under beam conditions

This section provides the total mass (M) of the virtual project with the use of different optimal sets for beam conditions, as well as the unfactored total mass ($\bar{M}$).

As mentioned in the other chapters, two different sets of design parameters (design spaces) and popularity weighting factors were used to obtain the final optimal sets for beam parameters, namely Weighting A for Design Space A (2686 data points) and Weighting B for Design Space B (3560 data points).

The total mass of the final optimal sets obtained for the different design spaces cannot be compared directly to each other, as the size of each design space and corresponding weighting factors, are different. Nevertheless, the relative competitiveness of each optimal set of sections can be investigated.

9.3.1 Final optimal sets for Design Space A

9.3.1.1 Based on Initial Set 1

Table 9.1 presents the comparison information of the final A1 Optimal Sets, considering all the data points from Design Space A (2686 data points).

As mentioned in Chapter 8, the optimal sets obtained from Initial Set 1 (A1 Optimal Sets) were not as sensitive to the different minimum ratings as the other optimal sets. This is shown in the comparison in Table 9.1. The total mass factored and unfactored were very close to each other, the difference between the different sets being close to 1 %.

The most optimal set of the six different final A1 Optimal Sets was found to be the optimal set obtained with a minimum rating of 80. This optimal set can be found in Table E.2 of

Appendix E. The optimal set with a minimum rating of 100 was not considered, as it contained too many sections (more than 80 sections).

Table 9.1: Comparison information of final A1 Optimal Sets with 2686 data points

Minimum rating of set	Number of sections in set	Total mass (tons)		% difference from lightest set	
		$\bar{M}$	M	$\bar{M}$	M
75	55	808	374	1.38	1.17
80	61	800	371	0.42	0.29
85	57	808	375	1.44	1.47
90	53	804	373	0.95	0.92
95	56	800	372	0.49	0.48
100	82	797	370	0.00	0.00

The most optimal set mentioned above was found to be more economical mainly because it contained more sections than any of the other sets from Initial Set 1. With an initial set which is not really sensitive to the minimum rating, the number of sections makes a big difference.

9.3.1.2 Based on Initial Set 2

Table 9.2 presents the comparison information of the final A2 Optimal Sets, considering 2683 of the data points from Design Space A, as the optimal sets obtained with a minimum rating of 75 and 80 did not cover all the data points. The data points which were not covered by the optimal sets were not popular and thus not important.

Table 9.2: Comparison information of final A2 Optimal Sets with 2683 data points

Minimum rating of set	Number of sections in set	Total mass (tons)		% difference from lightest set	
		$\bar{M}$	M	$\bar{M}$	M
75	41	659	308	7.27	6.02
80	41	651	306	5.87	5.12
85	52	642	303	4.42	4.08
90	60	632	298	2.83	2.48
95	118	623	295	1.44	1.27
100	525	615	291	0.00	0.00

The optimal sets obtained from Initial Set 2 (A2 Optimal Sets) were more sensitive to the different minimum ratings than the A1 Optimal Sets, as shown in Table 9.2, with the total mass difference as high as 6 % from the lightest sections (highlighted blue in Table 9.2).

The most optimal set among the six different final A2 Optimal Sets was found to be the set that was obtained with a minimum rating of 90. This set can be found in Table E.5 of Appendix E. The optimal sets with a minimum rating of 100 and 95 were not considered, as they contained too many sections (more than 80 sections).

9.3.1.3 Based on Initial Set 3

Table 9.3 presents the comparison information of the final A3 Optimal Sets, considering only 2679 of the data points from Design Space A, as the optimal set obtained with a minimum rating of 75 did not cover all the data points. The data points which were not covered were not popular and thus not important.

Table 9.3: Comparison information of final A3 Optimal Sets with 2679 data points

Minimum rating of set	**Number of sections in set**	**Total mass (tons)**		**% difference from lightest set**	
		$\bar{M}$	M	$\bar{M}$	M
75	50	643	305	3.03	2.48
80	51	644	306	3.28	2.60
85	56	637	302	2.11	1.50
90	72	629	300	0.88	0.71
95	79	628	300	0.64	0.59
100	170	624	298	0.00	0.00

The optimal sets obtained from Initial Set 3 (A3 Optimal Sets) were sensitive to the different minimum ratings, but not as sensitive as the A2 Optimal Sets, as shown in Table 9.3. The mass difference between the sets was never much more than 3 %.

The most optimal set between the six different final A3 Optimal Sets was found to be the optimal set obtained with a minimum rating of 95. This set can be found in Table E.8 of Appendix E. The optimal set with a minimum rating of 100 was not considered, as it contained too many sections (more than 80 sections).

9.3.1.4 Based on other I-section sets

Seven different sets of I-sections, available globally, were put through the same comparison methodology, as discussed in Section 9.2. The results of these sets are shown in Table 9.4. These sets included the European, British, American, Russian, Japanese and South African hot-rolled I-sections. It also included the South African hot-rolled I-sections combined with the plate girders used in South Africa. All of the globally available hot-rolled I-sections can be obtained from ArcelorMittal's website (ArcelorMittal, 2016) and the South African I-section from Appendix A.

Table 9.4: Comparison information of other sets with 2686 data points

Set description	Number of sections in set		Total mass (tons)	
	Start	**After reduction**	$\bar{M}$	M
SA hot-rolled	63	48	925	412
SA hot-rolled plus plate girders	155	47	840	397
European hot-rolled	321	60	840	397
British hot-rolled	208	58	853	395
American hot-rolled	292	50	807	376
Russian hot-rolled	38	28	941	429
Japanese hot-rolled	42	21	884	414

It was found that the American hot-rolled I-sections performs the best under the design parameters of Design Space A, as shown in Table 9.4. The optimal sets of welded I-sections, however, only had to outperform the I-sections currently used in South Africa.

The hot-rolled I-sections used in South Africa were found not to be effective over the entire Design Space A, as the South African hot-rolled I-sections, combined with the plate girders, outperformed the South African hot-rolled I-sections on its own. The combined set of hot-rolled I-sections and plate girders which worked for Design Space A and B can be found in Table F.1 of Appendix F. There was only one extra section needed for Design Space B.

9.3.2 Final optimal sets for Design Space B

9.3.2.1 Based on Initial Set 1

Table 9.5 presents the comparison information of the final B1 Optimal Sets, considering only 3559 of the data points from Design Space B, as most of the optimal sets did not cover all the data points. The data points which were not covered by these sets were not popular and thus not important.

As discussed in Chapter 8, the optimal sets obtained from Initial Set 1 were not as sensitive to the different minimum ratings as the other optimal sets, as shown in Table 9.5.

The most optimal set between the six different final B1 Optimal Sets was found to be the optimal set obtained with a minimum rating of 95, as it was the lightest set with a size less than 80 sections. This set can be found in Table E.3 of Appendix E.

The sizes of the B1 Optimal Sets were closer to each other than that of the A1 Optimal Sets. For this reason the most optimal set of the final B1 Optimal Set was found to be closer to the lightest optimal set, than the final A1 Optimal Sets.

Table 9.5: Comparison information of final B1 Optimal Sets with 3559 data points

Minimum rating of set	**Number of sections in set**	**Total mass (tons)**		**% difference from lightest set**	
		$\bar{M}$	M	$\bar{M}$	M
75	55	963	47.44	1.30	1.58
80	58	957	47.19	0.66	1.06
85	59	964	47.58	1.46	1.88
90	56	960	47.29	1.02	1.26
95	57	955	46.95	0.47	0.53
100	85	950	46.70	0.00	0.00

9.3.2.2 Based on Initial Set 2

Table 9.6 presents the comparison information of the final B2 Optimal Sets, considering all of the data points from Design Space B (3560 data points).

Table 9.6: Comparison information of final B2 Optimal Sets with 3560 data points

Minimum rating of set	**Number of sections in set**	**Total mass (tons)**		**% difference from lightest set**	
		$\bar{M}$	M	$\bar{M}$	M
75	47	782	40.40	6.61	6.25
80	47	780	40.20	6.32	5.72
85	58	771	39.75	5.20	4.54
90	72	754	38.98	2.86	2.51
95	119	746	38.55	1.76	1.37
100	601	733	38.03	0.00	0.00

As discussed in Chapter 8, the optimal sets obtained from Initial Set 2 were much more sensitive to the different minimum ratings than the optimal sets obtained from Initial Set 1, as shown in Table 9.6.

The most optimal set between the six different final B2 Optimal Sets, with a size less than 80 sections, was found to be the one obtained with a minimum rating of 90. This set can be found in Table E.6 of Appendix E. The minimum rating of the most optimal set corresponds to the minimum rating of the most optimal set obtained from the A2 Optimal Sets.

9.3.2.3 Based on Initial Set 3

Table 9.7 presents the comparison information of the final B3 Optimal Sets, considering all of the data points from Design Space B (3560 data points).

Table 9.7: Comparison information of final B3 Optimal Sets with 3560 data points

Minimum rating of set	Number of sections in set	Total mass (tons)		% difference from lightest set	
		$\bar{M}$	M	$\bar{M}$	M
75	56	776	40.29	3.74	3.44
80	61	771	40.14	3.00	3.05
85	61	768	39.95	2.60	2.57
90	65	761	39.38	1.71	1.10
95	85	754	39.12	0.82	0.42
100	187	748	38.95	0.00	0.00

As discussed in Chapter 8, the optimal sets obtained from Initial Set 3 were also sensitive to the different minimum ratings, but not as sensitive as the optimal sets obtained from Initial Set 2, as shown in Table 9.7.

The most optimal set between the six different final B3 Optimal Sets was found to be the optimal set obtained with a minimum rating of 90, as it was the lightest set, with a size less than 80 sections. This optimal set can be found in Table E.9 of Appendix E.

9.3.2.4 Based on other I-section sets

Table 9.8 presents the comparison information of the other sets of I-sections available globally. The South African hot-rolled I-sections only worked for 3554 of the 3560 data points from Design Space B, as shown in Table 9.8.

Table 9.8: Comparison information of other sets with 3554 data points

Set description	Number of sections in set		Total mass (tons)		Number of data points
	Start	After reduction	$\bar{M}$	M	
SA hot-rolled	63	48	1105	50.67	3554
SA hot-rolled plus plate girders	155	48	993	48.58	3560
European hot-rolled	321	62	998	47.69	3560
British hot-rolled	208	59	1015	49.12	3560
American hot-rolled	292	53	963	46.48	3560
Russian hot-rolled	38	28	1132	52.22	3560
Japanese hot-rolled	42	22	1056	50.80	3560

The set of American hot-rolled I-sections was found to perform the best under the design parameters of Design Space B, which is the same as for Design Space A.

As found for Design Space A (refer to Section 9.3.1.4), the hot-rolled I-sections used in South Africa were also found to be ineffective over the entire Design Space B. The dimensions of the South African hot-rolled I-sections and plate girders that worked for Design Space B can be found in Table F.1 of Appendix F.

9.3.3 Comparison of optimal sets

This section compares the most optimal sets obtained from Initial Sets 1, 2 and 3 with the different sets of I-sections available in South Africa.

The design parameters and weighting factors of Design Space B (Weighting B) were regarded as being representative of the actual design parameters, ruling in the South African steel construction industry. Accordingly, the best optimal set was chosen based on the comparison results of the optimal sets under the design parameters of Design Space B.

The most optimal sets were first compared to the South African hot-rolled I-sections and plate girders, together in one set, after which the most optimal sets were compared to the South African hot-rolled I-sections on their own, with and without IPE's, but limited to only the part of the design space over which they are effective, as it was found in Section 9.3.1.4 and 9.4.1.2 that the South African hot-rolled I-sections on their own are not effective over the entire Design Space A and B.

9.3.3.1 Compared to South African hot-rolled I-sections and plate girders

It was found that the total mass (M), of an average project under Weighting B, will reduce by approximately 3.5 % and the unfactored total mass ($\bar{M}$) by approximately 3 % when the most optimal set from Initial Set 1 is used instead of the currently used I-sections in South Africa, as shown in Table 9.9. This means that the cost of steelwork can already be reduced slightly by just using welded I-sections in the image of the hot-rolled I-sections available globally.

When welded I-sections with more optimal cross-sections (optimal sets from Initial 2 and 3) are used instead of the currently used I-sections in South Africa, the total mass can be reduced by about 20 %. It will also reduce the unfactored total mass by some 23 %, as shown in Table 9.9.

The total mass difference between the most optimal set of Initial Sets 2 and 3 is marginal, at less than 1 %. A optimal set of welded I-sections obtained from Initial Set 2 (created with smaller dimensional increments) is thus not justified. The optimal set obtained from Initial Set 3 was therefore considered to be the best set for beam parameters, practically and economically. The same conclusions can also be drawn from Table 9.10.

When taking into account that the South African IPE's can be used in combination with the most optimal set of Initial Set 3, as they are still being produced in South Africa, the total mass can be reduced by almost 21 %, as shown in Table 9.9. The IPE's, in combination with the optimal set of welded I-sections, were able to reduce the weight of the virtual project even further, as it is not possible to fabricate welded I-sections which are lighter than the South African IPE sections, as mentioned in Section 2.2.3. It was also noted that the IPE's do not replace any of the welded I-sections when they are used in combination.

Table 9.9: Comparison between best optimal sets to South African I-sections under Weighting B

Set description	Number of sections in set	Total mass (tons)		% difference from SA I-sections	
		$\bar{M}$	M	$\bar{M}$	M
Hot-rolled SA plus plate girders	48	993	48.58	0.00	0.00
Best B1 Optimal Set	57	956	46.95	-3.68	-3.37
Best B2 Optimal Set	72	754	38.98	-24.04	-19.77
Best B3 Optimal Set	65	761	39.38	-23.34	-18.94
Best B3 Optimal Set plus IPE's	74	753	38.58	-24.14	-20.59

The percentage difference of the optimal sets to the South African I-sections was found to be not much different from Design Space A to B, as shown in Table 9.10. The comparison methodology is thus not sensitive to different popularity weighting factors, as is also the case with the optimisation methodology presented in Chapter 5.

The best B3 Optimal Set was also put through the comparison methodology corresponding to Design Space A in order to determine how sensitive the total mass results are to change in popularity weighting factors. This could, however, not be done with the best A3 Optimal Set and Design Space B, as the best A3 Optimal Set did not work for all the design parameters of Design Space B.

It was found that the total mass of the two optimal sets differ less than 1 % under the same design space (Design Space A) and popularity weighting factors (Weighting A). The best B3 Optimal Set only needed 63 sections to cover Design Space A, as shown in Table 9.10. The end result of total mass is thus not sensitive to a change in popularity weighting factors, but could be sensitive to the design parameter ranges when they differ a lot.

Table 9.10: Comparison between best optimal sets to South African I-sections under Weighting A

Set description	Number of sections in set	Total mass (tons)		% difference from SA I-sections	
		$\bar{M}$	M	$\bar{M}$	M
Hot-rolled SA plus plate girders	47	840	397	0.00	0.00
Best A1 Optimal Set	61	800	371	-4.81	-6.71
Best A2 Optimal Set	60	635	298	-24.42	-25.02
Best A3 Optimal Set	79	635	300	-24.40	-24.59
Best A3 Optimal Set plus IPE's	86	631	299	-24.90	-24.69
Best B3 Optimal Set	63	641	302	-23.69	-23.93

9.3.3.2 Compared to South African hot-rolled I-sections

As mentioned at the start of Section 9.3.3, the South African hot-rolled I-sections were not effective over the entire Design Space A and B. This means that only the design parameters, for which the South African hot-rolled I-sections were effective, were accounted for.

The South African hot-rolled I-sections were only effective at 2335 of the 2686 data points of Design Space A and only effective for 3099 of the 3560 data points of Design Space B. The total mass of these data points, with and without weighting factors, for the optimal sets of welded I-sections and South African hot-rolled I-sections are presented in Tables 9.12 and 9.11.

Tables 9.11 and 9.12 lead to the same conclusions as were determined in Section 9.3.3.1 with respect to the existing South African hot-rolled I-sections. This means that if only the hot-rolled I-sections in South Africa are replaced with welded I-sections, the total mass of the sections used in the structures in the country will reduce by 18 %. Should the welded I-sections be used in combination with the available IPE's, the total mass will reduce by 20 % (refer to Table 9.11).

Table 9.11: Comparison between best optimal sets to South African hot-rolled I-sections under Weighting B

Set description	**Total mass (tons)**		**% difference from Hot-rolled SA**	
	$\bar{M}$	M	$\bar{M}$	M
Hot-rolled SA	588	37.55	0.00	0.00
Best B1 Optimal Set	558	36.46	-5.09	-2.91
Best B2 Optimal Set	447	30.50	-24.03	-18.76
Best B3 Optimal Set	450	30.77	-23.37	-18.05
Best B3 Optimal Set plus IPE's	442	29.97	-24.72	-20.19

Table 9.12: Comparison between best optimal sets to South African hot-rolled I-sections under Weighting A

Set description	**Total mass (tons)**		**% difference from Hot-rolled SA**	
	$\bar{M}$	M	$\bar{M}$	M
Hot-rolled SA	532	308	0.00	0.00
Best A1 Optimal Set	501	286	-5.81	-7.11
Best A2 Optimal Set	402	230	-24.46	-25.36
Best A3 Optimal Set	403	232	-24.15	-24.75
Best A3 Optimal Set plus IPE's	399	232	-24.94	-24.88

9.3.3.3 Compared to South African hot-rolled I-sections without IPE's

It is clear from the results in Section 9.3.3.2 that welded I-sections can not outperform IPE's, but that they perform really well when compare to the other hot-rolled I-sections available in South Africa.

This section determines by which margin the optimal sets of welded I-sections outperform the South African hot-rolled I-sections, excluding IPE's.

The South African hot-rolled I-sections, excluding IPE's, were only effective at 1540 of the 2686 data points of Design Space A and effective at 1720 of the 3560 data points of Design Space B.

Tables 9.13 and 9.14 lead to the same conclusions as were determined in Section 9.3.3.1 with respect to the existing South African hot-rolled I-sections, excluding IPE's. The optimal sets of welded I-sections outperforms the South African hot-rolled I-sections (IPE's excluded) with a larger margin than originally expected. The total mass of the optimal sets of welded I-sections were almost 23 % lighter than the corresponding hot-rolled I-sections over the defined region of 1720 data points for Design Space B, as shown in Table 9.13.

Table 9.13: Comparison between best optimal sets to South African hot-rolled I-sections, excluding IPE's, under Weighting B

Set description	**Total mass (tons)**		**% difference from Hot-rolled SA**	
	$\bar{M}$	M	$\bar{M}$	M
Hot-rolled SA (excluding IPE's)	542	32.37	0.00	0.00
Best B1 Optimal Set	501	30.03	-7.54	-7.23
Best B2 Optimal Set	394	24.70	-24.03	-23.69
Best B3 Optimal Set	399	25.09	-23.37	-22.49

Table 9.14: Comparison between best optimal sets to South African hot-rolled I-sections, excluding IPE's, under Weighting A

Set description	**Total mass (tons)**		**% difference from Hot-rolled SA**	
	$\bar{M}$	M	$\bar{M}$	M
Hot-rolled SA (excluding IPE's)	491	295	0.00	0.00
Best A1 Optimal Set	453	273	-7.76	-7.53
Best A2 Optimal Set	359	217	-26.91	-26.21
Best A3 Optimal Set	362	219	-26.42	-25.59

9.4 Comparison of final optimal sets under girder conditions

This section provides the total mass (M) of the virtual project with the use of different optimal sets under girder conditions, as well as the unfactored total mass ($\bar{M}$).

As mentioned in the preceding chapters, one set of design parameters (Design Space C) and one set of popularity weighting factors (Weighting C) were used to obtain the final optimal sets for girders. Design Space C contained 18,060 different data points.

9.4.1 Final optimal sets for Design Space C

9.4.1.1 Based on Initial Set 4

Table 9.15 presents the comparison information of the final C4 Optimal Sets, considering only 16,312 data points of Design Space C (18,060 data points), as the final C4 Optimal Set obtained with a minimum rating of 100, only worked for 16,312 of the data points of Design Space C. The data points not covered were not popular and thus not important.

The C4 Optimal Sets with a minimum rating of 75 and 80 were abandoned, as these sets had too many sections after Step 5 of the optimisation methodology (discussed in Section 8.3).

Table 9.15: Comparison information of final C4 Optimal Sets at 16,312 data points

Minimum rating of set	Number of section in set	Total mass (tons)		% difference from lightest set	
		$\bar{M}$	M	$\bar{M}$	M
85	140	11443	45.67	2.54	3.17
90	254	11310	45.10	1.34	1.87
95	403	11166	44.49	0.06	0.51
100	777	11160	44.27	0.00	0.00

The most optimal set between the four different final C4 Optimal Sets was found to be the optimal set obtained with a minimum rating of 85, as it was the lightest set, with a size less than 150 sections. This optimal set can be found in Table E.12 of Appendix F.

9.4.1.2 Based on other I-section sets

Table 9.16 presents the comparison information of the globally available sets of I-sections. The South African I-sections (hot-rolled I-sections and plate girders) only worked for 18,049 of the 18,060 data points from Design Space C, as shown in Table 9.16.

Table 9.16: Comparison information of other sets

Set description	**Number of sections in set**		**Total mass (tons)**		**Number of data points**
	Start	**After reduction**	$\bar{M}$	M	
SA hot-rolled plus plate girders	155	71	18151	55.08	18049
European hot-rolled	321	73	22710	61.93	18060
British hot-rolled	208	67	22385	59.53	18060
American hot-rolled	292	79	22324	59.11	18060
Russian hot-rolled	38	27	8019	64.43	10520
Japanese hot-rolled	42	27	13390	64.86	15078

The set of South African hot-rolled I-section and plate girder combined was found to perform the best under the design parameters of Design Space C. This South African set out performed the other sets, as Design Space C contained design parameters with high loads over larger spans, where the plate girders performed better than the hot-rolled I-sections. The dimensions of these sections that worked for Design Space C can be found in Table F.1 of Appendix F.

It was found that some IPE's worked for some of the light design parameters, which meant that some of the design parameters in Design Space C were loaded too low and should not have been included in the design space, as IPE's are never used as girders.

9.4.2 Comparison of optimal sets

This section compares the most optimal sets obtained from Initial Set 4 with the different I-sections available in South Africa, within Design Space C.

The most optimal sets were first compared to the South African hot-rolled I-sections and plate girders, together in one set, after which the most optimal sets were compared to the South African hot-rolled I-sections and plate girder on its own, but only over the part of the design space where they are effective.

9.4.2.1 Compared to South African hot-rolled I-sections and plate girders

The sets of I-sections were compared with 17,077 of the 18,060 data points, as these were the only data points which corresponded between the South African I-sections and the best C4 Optimal Set. The data points which were not covered by these sets were, however, not popular and thus not important.

It was found that the total mass with weightings (M) of an average project under Weighting C will reduce by approximately 17 % and the unfactored total mass ($\bar{M}$) by 12 %, when the best C4 Optimal Set is used instead of the currently used I-sections in South Africa, as shown in Table 9.17.

Table 9.17: Comparison between best optimal sets to South African I-sections under Weighting C

Set description	Number of sections in set	Total mass (tons)		% difference from SA I-sections	
		$\bar{M}$	M	$\bar{M}$	M
Hot-rolled SA plus plate girders	71	14976	55.08	0.00	0.00
Best C4 Optimal Set	140	13218	45.67	-11.74	-17.08

9.4.2.2 Compared to South African hot-rolled I-sections

The South African hot-rolled I-sections were not effective over the entire design space and was therefore only compared at 4963 of the 17,077 data points of Design Space C. The total mass of these data points, with and without weighting factors, for the optimal sets of welded I-sections and South African hot-rolled I-sections are presented in Table 9.18.

It was found that the total mass with weightings of an average project under Weighting C will reduce by approximately 19 % and the unfactored total mass ($\bar{M}$) by 22 % when the best C4 Optimal Set is used instead of the currently used hot-rolled I-sections in South Africa, as shown in Table 9.18.

Table 9.18: Comparison between best optimal sets to South African hot-rolled I-sections under Weighting C

Set description	Total mass (tons)		% difference from Hot-rolled SA	
	$\bar{M}$	M	$\bar{M}$	M
Hot-rolled SA	1154	22.81	0.00	0.00
Best C4 Optimal Set	897	18.47	-22.29	-19.02

9.4.2.3 Compared to South African plate girders

The South African plate girders were not effective over the entire design space and was therefore only compared at 12,114 of the 17,077 data points of Design Space C. The total mass of these data points, with and without weighting factors, for the optimal sets of welded I-sections and South African plate girders are presented in Table 9.19.

It was found that the total mass with weightings (M) of an average project under Weighting C will reduce by approximately 16 % and the unfactored total mass ($\bar{M}$) by 11 % when the best C4 Optimal Set is used instead of the currently used plate girders in South Africa, as shown in Table 9.19.

Table 9.19: Comparison between best optimal sets to South African plate girders under Weighting C

Set description	Total mass (tons)		% difference from SA plate girders	
	$\bar{M}$	M	$\bar{M}$	M
SA plate girders	13822	32.28	0.00	0.00
Best C4 Optimal Set	12321	27.21	-10.86	-15.71

9.5 Best optimal sets

9.5.1 Best optimal set of welded I-sections for beam parameters

As mentioned in Section 9.3.3, the most optimal set deriving from Initial Set 3 for Design Space B was found to be the best optimal set of welded I-sections for beam parameters. This set of welded I-sections can be found in Table E.9 of Appendix E. The sections not highlighted in this table can be used to replace South African hot-rolled I-sections and the sections highlighted can be used to replace the South African plate girders for Design Space B.

About 78 % of the sections in the best B3 Optimal Set also occurred in the A3 Optimal Set with the same minimum rating of 90. The different sets of popularity weighting factors do therefore not make a large difference to the sections present in the optimal set. Varying the minimum rating did however make a difference, as the best optimal set only corresponded 57 % with the A3 Optimal Set obtained with a minimum rating of 95.

Some of the sections found in the best optimal set were quite slender, with a web slenderness (h_w^*/t_w) of 170, as shown in Table E.9. This exceed the maximum web slenderness ratio of 155 of the plate girders presented in the Red Book (SAISC, 2013) and the 160 practical web slenderness, as specified by De Clercq (2010). However, if the best optimal set is not practical for steel fabricators, then the B3 Optimal Set obtained with a minimum rating of 80, presented in Table E.10 of Appendix E, can be used as an alternative. The sections in this set have a maximum web slenderness of 160 and the set is only 2 % heavier according to the comparison methodology, as shown in Table 9.7.

The final ranking in Tables E.9 and E.10 corresponds to the final ranking obtained in Step 7 of the optimisation methodology (refer to Chapter 5) and can be used to determine how well the sections perform against each other in Design Space B.

The web to flange thickness ratio (t_w/t_f) of the optimal sets were never smaller than 0.3. This is close to the minimum web to flange thickness ratio of 0.27 of the plate girders presented in the Red Book (SAISC, 2013), even though a minimum web to flange ratio of 0.17 was specified in the creation of the welded I-sections (refer to Section 4.4.1).

The web to total area ratio (A_w/A) of the welded I-sections varied a lot. When the thickness of the flange was small compared to the I-section depth, the web to total area ratio (A_w/A) was also not always in the region of 0.63, which corresponds to the practical web to total area ratios derived by Schilling (1974) (refer to Section 2.7.2.1). This was, however, more the case for the larger welded I-sections.

9.5.2 Best optimal set of welded I-sections for girder parameters

As mentioned in Section 9.4.2.1, the most optimal set deriving from Initial Set 4 for Design Space C was found to be the best optimal set of welded I-sections for girder parameters. This set of welded I-sections can be found in Table E.12 of Appendix E. The sections not highlighted can be used to replace South African plate girders and the sections highlighted can be used to replace the South African hot-rolled I-sections for Design Space C.

A large number of the girders have a web slenderness (h_w^*/t_w) more than 160, as shown in Table E.12. Some 37 (26 %) of the 140 I-sections have a web slenderness more than 160, as the maximum web slenderness specified in SANS 10162-1 (SABS, 2011c) is 233.8. All of these girders are deeper than 850 mm and can thus be stiffened where needed.

The web to flange thickness ratios (t_w/t_f) of the optimal set was never smaller than 0.28, which is also close to the minimum web to flange thickness ratio of 0.27 of the plate girders presented in the Red Book (SAISC, 2013). The web to total area ratio (A_w/A) did also not always correspond to the practical web to total area ratios derived by Schilling (1974) (refer to Section 2.7.2.1).

9.5.3 Best optimal set of welded I-sections for beams and girders

The best optimal set of beams and girders can be obtained by combining the best optimal set of beams, from Section 9.5.1, with the best optimal set of girders, from Section 9.5.2. This set contains 199 welded I-sections, which is 44 sections more than the set of I-sections (including hot-rolled I-sections and plate girders) currently used in South Africa. The best optimal set of beams and girders can however be reduced by the use of the optimisation methodology in Chapter 9.2.

The set of 199 beams and girders were put through the optimisation methodology for beams (Design Space B) and girders (Design Space C) to reduce it to 150 sections. This was achieved with the use of an allowable overlap percentage of 100 % to 80 % in the optimisation process (refer to Chapter 9.2).

The 199 I-sections were reduced to 46 sections in the optimisation process for beams and to 125 sections in the optimisation process for girders. These sections combined to give an optimal set of 149 sections for the use as beams and girders. This set is provided in Table 9.22. When a section does not have a final ranking in Table 9.22, it was not effective over the corresponding design space.

The reduced set of I-sections are almost 17 % lighter than the South African hot-rolled I-sections and plate girders under beam conditions, as shown in Table 9.20. This is only approximately

2 % less than what can be obtained with the most optimal set of welded I-sections for beam parameters (refer to Table 9.9).

Table 9.20: Comparison between the reduced optimal sets and the South African I-sections under Weighting B

Set description	**Number of sections in set**	**Total mass (tons)**		**% difference from SA I-sections**	
		$\bar{M}$	M	$\bar{M}$	M
Hot-rolled SA plus plate girders	48	993	48.58	0.00	0.00
Reduced optimal set	46	780	40.45	-21.45	-16.74

The reduced set of I-sections are also almost 17 % lighter than the South African hot-rolled I-sections and plate girders under girder conditions, as shown in Table 9.21. This is almost the same as what can be obtain with the most optimal set of welded I-sections for girder parameters (refer to Table 9.17).

The total mass presented in Table 9.21 is only for 16,774 data points, as the reduced optimal set only works for these design parameters. The design parameters not covered are not popular and thus not imported.

Table 9.21: Comparison between the reduced optimal sets and the South African I-sections under Weighting C

Set description	**Number of sections in set**	**Total mass (tons)**		**% difference from SA I-sections**	
		$\bar{M}$	M	$\bar{M}$	M
Hot-rolled SA plus plate girders	71	14191	55.08	0.00	0.00
Reduced optimal set	125	12610	45.74	-11.14	-16.96

Table 9.22: The reduced optimal set of welded I-sections

Section Name $h_w^* \times b \times m$	**Final ranking (B)**	**Final ranking (C)**	h_w^* **mm**	b **mm**	t_w **mm**	t_f **mm**	t_w/t_f	A_w/A	h_w^*/t_w
150.0 x 120.0 x 17.19	1408.31		150	120	5	6	0.83	0.34	30
150.0 x 120.0 x 20.96	564.41		150	120	5	8	0.63	0.28	30
200.0 x 100.0 x 15.7	11968.21	510.77	200	100	5	5	1.00	0.50	40
200.0 x 120.0 x 19.15	462.83	155.06	200	120	5	6	0.83	0.41	40
200.0 x 120.0 x 22.92	411.82	163.16	200	120	5	8	0.63	0.34	40
200.0 x 140.0 x 25.43	860.23	132.72	200	140	5	8	0.63	0.31	40
200.0 x 160.0 x 27.95	773.39		200	160	5	8	0.63	0.28	40
250.0 x 100.0 x 17.66	780.15	349.56	250	100	5	5	1.00	0.56	50

Table 9.22 (continued)

Section Name $h_w^* \times b \times m$	**Final ranking (B)**	**Final ranking (C)**	h_w^* **mm**	b **mm**	t_w **mm**	t_f **mm**	t_w/t_f	A_w/A	h_w^*/t_w
250.0 x 120.0 x 21.12	20.02		250	120	5	6	0.83	0.46	50
250.0 x 120.0 x 23.08		8.34	250	120	6	6	1.00	0.51	42
250.0 x 120.0 x 28.65		15.13	250	120	5	10	0.50	0.34	50
250.0 x 140.0 x 27.4		298.19	250	140	5	8	0.63	0.36	50
250.0 x 160.0 x 29.91	246.14		250	160	5	8	0.63	0.33	50
250.0 x 160.0 x 34.93	581.89	93.87	250	160	5	10	0.50	0.28	50
300.0 x 100.0 x 19.62	647.26	367.75	300	100	5	5	1.00	0.60	60
300.0 x 100.0 x 21.2		430.54	300	100	5	6	0.83	0.56	60
300.0 x 120.0 x 25.43		19.66	300	120	6	6	1.00	0.56	50
300.0 x 140.0 x 31.71		10.81	300	140	6	8	0.75	0.45	50
300.0 x 140.0 x 36.11		11.17	300	140	6	10	0.60	0.39	50
300.0 x 160.0 x 31.87	241.60	109.44	300	160	5	8	0.63	0.37	60
300.0 x 180.0 x 40.03	674.52	114.46	300	180	5	10	0.50	0.29	60
300.0 x 200.0 x 43.17	668.00	106.73	300	200	5	10	0.50	0.27	60
300.0 x 200.0 x 49.45	685.60		300	200	5	12	0.42	0.24	60
300.0 x 220.0 x 53.22	578.87		300	220	5	12	0.42	0.22	60
300.0 x 240.0 x 64.53	252.58		300	240	5	14	0.36	0.18	60
350.0 x 100.0 x 21.59	575.78		350	100	5	5	1.00	0.64	70
350.0 x 100.0 x 23.16		430.60	350	100	5	6	0.83	0.59	70
350.0 x 100.0 x 29.05		6.67	350	100	6	8	0.75	0.57	58
350.0 x 120.0 x 25.04	10.07		350	120	5	6	0.83	0.55	70
350.0 x 140.0 x 31.32		710.94	350	140	5	8	0.63	0.44	70
350.0 x 260.0 x 70.89	336.83		350	260	5	14	0.36	0.19	70
350.0 x 280.0 x 75.28	257.49		350	280	5	14	0.36	0.18	70
400.0 x 100.0 x 23.55	504.74		400	100	5	5	1.00	0.67	80
400.0 x 100.0 x 25.12		460.87	400	100	5	6	0.83	0.63	80
400.0 x 100.0 x 28.26		481.51	400	100	5	8	0.63	0.56	80
400.0 x 120.0 x 37.68		37.50	400	120	6	10	0.60	0.50	67
400.0 x 140.0 x 33.28		200.01	400	140	5	8	0.63	0.47	80

Table 9.22 (continued)

Section Name $h_w^* \times b \times m$	**Final ranking (B)**	**Final ranking (C)**	h_w^* mm	b mm	t_w mm	t_f mm	t_w/t_f	A_w/A	h_w^*/t_w
400.0 x 160.0 x 40.82		130.81	400	160	5	10	0.50	0.38	80
400.0 x 160.0 x 43.96		78.83	400	160	6	10	0.60	0.43	67
400.0 x 180.0 x 47.1		78.93	400	180	6	10	0.60	0.40	67
400.0 x 240.0 x 60.92	724.23		400	240	5	12	0.42	0.26	80
450.0 x 100.0 x 25.51	431.48		450	100	5	5	1.00	0.69	90
450.0 x 140.0 x 35.25		664.21	450	140	5	8	0.63	0.50	90
450.0 x 160.0 x 37.76	70.82	358.19	450	160	5	8	0.63	0.47	90
450.0 x 160.0 x 46.31		44.17	450	160	6	10	0.60	0.46	75
450.0 x 180.0 x 49.45		25.64	450	180	6	10	0.60	0.43	75
450.0 x 180.0 x 55.11		41.95	450	180	6	12	0.50	0.38	75
450.0 x 200.0 x 49.06		418.05	450	200	5	10	0.50	0.36	90
450.0 x 200.0 x 52.59		25.84	450	200	6	10	0.60	0.40	75
450.0 x 220.0 x 62.64		72.93	450	220	6	12	0.50	0.34	75
500.0 x 100.0 x 27.48	378.61		500	100	5	5	1.00	0.71	100
500.0 x 160.0 x 39.72	157.75	406.95	500	160	5	8	0.63	0.49	100
500.0 x 220.0 x 65		38.99	500	220	6	12	0.50	0.36	83
500.0 x 320.0 x 100.01	100.44		500	320	5	16	0.31	0.20	100
550.0 x 120.0 x 32.89		253.88	550	120	5	6	0.83	0.66	110
550.0 x 180.0 x 62.8		32.23	550	180	8	10	0.80	0.55	69
550.0 x 220.0 x 67.35		40.14	550	220	6	12	0.50	0.38	92
550.0 x 240.0 x 66.8	126.33	89.06	550	240	5	12	0.42	0.32	110
550.0 x 280.0 x 83.13	248.84		550	280	5	14	0.36	0.26	110
600.0 x 100.0 x 31.4	564.37	273.40	600	100	5	5	1.00	0.75	120
600.0 x 100.0 x 36.11	68.94	49.87	600	100	5	8	0.63	0.65	120
600.0 x 120.0 x 34.85	92.38		600	120	5	6	0.83	0.68	120
600.0 x 140.0 x 41.13		325.65	600	140	5	8	0.63	0.57	120
600.0 x 160.0 x 43.65	100.10	488.69	600	160	5	8	0.63	0.54	120
600.0 x 180.0 x 51.81	20.09	229.67	600	180	5	10	0.50	0.45	120

Table 9.22 (continued)

Section Name $h_w^* \times b \times m$	**Final ranking (B)**	**Final ranking (C)**	h_w^* **mm**	b **mm**	t_w **mm**	t_f **mm**	t_w/t_f	A_w/A	h_w^*/t_w
600.0 x 220.0 x 69.71		24.19	600	220	6	12	0.50	0.41	100
650.0 x 100.0 x 33.36	175.83		650	100	5	5	1.00	0.76	130
650.0 x 100.0 x 38.07	128.07		650	100	5	8	0.63	0.67	130
650.0 x 160.0 x 45.61	58.84	405.34	650	160	5	8	0.63	0.56	130
650.0 x 160.0 x 65.94		31.77	650	160	8	10	0.80	0.62	81
650.0 x 200.0 x 56.91	1.76		650	200	5	10	0.50	0.45	130
650.0 x 280.0 x 87.06		67.02	650	280	5	14	0.36	0.29	130
700.0 x 180.0 x 55.73	1.82	493.04	700	180	5	10	0.50	0.49	140
700.0 x 180.0 x 72.22		6.16	700	180	8	10	0.80	0.61	88
700.0 x 200.0 x 58.88	43.83	436.31	700	200	5	10	0.50	0.47	140
700.0 x 220.0 x 74.42		47.03	700	220	6	12	0.50	0.44	117
700.0 x 240.0 x 78.19		79.53	700	240	6	12	0.50	0.42	117
750.0 x 160.0 x 49.53		456.45	750	160	5	8	0.63	0.59	150
750.0 x 200.0 x 60.84		204.96	750	200	5	10	0.50	0.48	150
800.0 x 220.0 x 72.85		244.31	800	220	5	12	0.42	0.43	160
800.0 x 240.0 x 82.9		97.23	800	240	6	12	0.50	0.45	133
850.0 x 100.0 x 52.2	20.80		850	100	5	12	0.42	0.64	170
850.0 x 120.0 x 52.2	20.80	222.47	850	120	5	10	0.50	0.64	170
850.0 x 120.0 x 55.97		117.20	850	120	5	12	0.42	0.60	170
850.0 x 160.0 x 58.48		152.13	850	160	5	10	0.50	0.57	170
850.0 x 200.0 x 64.76		534.80	850	200	5	10	0.50	0.52	170
850.0 x 320.0 x 113.75	16.11		850	320	5	16	0.31	0.29	170
900.0 x 100.0 x 86.35		1.89	900	100	10	10	1.00	0.82	90
900.0 x 160.0 x 67.51		27.13	900	160	6	10	0.60	0.63	150
900.0 x 200.0 x 114.61		2.89	900	200	10	14	0.71	0.62	90
900.0 x 220.0 x 119.01		3.46	900	220	10	14	0.71	0.59	90
900.0 x 220.0 x 125.91		6.15	900	220	10	16	0.63	0.56	90
900.0 x 260.0 x 135.96		3.65	900	260	10	16	0.63	0.52	90

Table 9.22 (continued)

Section Name $h_w^* \times b \times m$	**Final ranking (B)**	**Final ranking (C)**	h_w^* mm	b mm	t_w mm	t_f mm	t_w/t_f	A_w/A	h_w^*/t_w
900.0 x 280.0 x 149.78		2.69	900	280	10	18	0.56	0.47	90
950.0 x 200.0 x 68.69		315.01	950	200	5	10	0.50	0.54	190
950.0 x 200.0 x 76.14		92.95	950	200	6	10	0.60	0.59	158
950.0 x 220.0 x 78.74		289.59	950	220	5	12	0.42	0.47	190
950.0 x 240.0 x 89.96		205.62	950	240	6	12	0.50	0.50	158
950.0 x 240.0 x 97.5		79.11	950	240	6	14	0.43	0.46	158
950.0 x 260.0 x 116.81		2.86	950	260	8	14	0.57	0.51	119
950.0 x 260.0 x 94.44	129.47	275.42	950	260	5	14	0.36	0.39	190
950.0 x 280.0 x 106.29		170.40	950	280	6	14	0.43	0.42	158
950.0 x 280.0 x 107.62		33.88	950	280	5	16	0.31	0.35	190
950.0 x 280.0 x 115.08		48.56	950	280	6	16	0.38	0.39	158
950.0 x 280.0 x 121.2		1.80	950	280	8	14	0.57	0.49	119
950.0 x 280.0 x 130		13.05	950	280	8	16	0.50	0.46	119
950.0 x 300.0 x 112.65		40.00	950	300	5	16	0.31	0.33	190
950.0 x 300.0 x 120.11		18.42	950	300	6	16	0.38	0.37	158
950.0 x 300.0 x 135.02		14.14	950	300	8	16	0.50	0.44	119
950.0 x 320.0 x 125.13		19.56	950	320	6	16	0.38	0.36	158
950.0 x 320.0 x 127.72		2.99	950	320	5	18	0.28	0.29	190
950.0 x 320.0 x 135.18		11.23	950	320	6	18	0.33	0.33	158
950.0 x 320.0 x 140.04		13.48	950	320	8	16	0.50	0.43	119
950.0 x 320.0 x 150.09		4.92	950	320	8	18	0.44	0.40	119
950.0 x 320.0 x 160.14		5.26	950	320	8	20	0.40	0.37	119
950.0 x 340.0 x 140.83		6.81	950	340	6	18	0.33	0.32	158
950.0 x 340.0 x 151.5		4.68	950	340	6	20	0.30	0.30	158
950.0 x 380.0 x 167.05		1.83	950	380	8	18	0.44	0.36	119
1050.0 x 200.0 x 87.14		124.40	1050	200	6	12	0.50	0.57	175
1050.0 x 220.0 x 147.27		5.77	1050	220	12	14	0.86	0.67	88
1100.0 x 160.0 x 81.95		11.28	1100	160	6	12	0.50	0.63	183
1100.0 x 240.0 x 95.93		60.29	1100	240	5	14	0.36	0.45	220
1100.0 x 320.0 x 166.73		4.00	1100	320	10	16	0.63	0.52	110
1150.0 x 180.0 x 73.4		117.28	1150	180	5	10	0.50	0.61	230

Table 9.22 (continued)

Section Name $h_w^* \times b \times m$	**Final ranking (B)**	**Final ranking (C)**	h_w^* **mm**	b **mm**	t_w **mm**	t_f **mm**	t_w/t_f	A_w/A	h_w^*/t_w
1200.0 x 280.0 x 155.74		10.12	1200	280	10	14	0.71	0.60	120
1200.0 x 300.0 x 169.56		3.37	1200	300	10	16	0.63	0.56	120
1250.0 x 240.0 x 131.25		8.83	1250	240	8	14	0.57	0.60	156
1250.0 x 280.0 x 140.04		20.11	1250	280	8	14	0.57	0.56	156
1250.0 x 280.0 x 159.67		4.18	1250	280	10	14	0.71	0.61	125
1300.0 x 240.0 x 106.45		58.15	1300	240	6	12	0.50	0.58	217
1300.0 x 240.0 x 113.98		41.24	1300	240	6	14	0.43	0.54	217
1300.0 x 260.0 x 118.38		31.59	1300	260	6	14	0.43	0.52	217
1300.0 x 320.0 x 182.43		13.95	1300	320	10	16	0.63	0.56	130
1350.0 x 280.0 x 133.92		29.87	1350	280	6	16	0.38	0.47	225
1350.0 x 280.0 x 146.32		18.85	1350	280	8	14	0.57	0.58	169
1350.0 x 300.0 x 160.14		19.19	1350	300	8	16	0.50	0.53	169
1350.0 x 360.0 x 165.32		7.68	1350	360	6	18	0.33	0.38	225
1400.0 x 260.0 x 123.09		24.32	1400	260	6	14	0.43	0.54	233
1400.0 x 320.0 x 146.32		18.88	1400	320	6	16	0.38	0.45	233
1450.0 x 280.0 x 152.6		12.27	1450	280	8	14	0.57	0.60	181
1500.0 x 320.0 x 198.13		14.55	1500	320	10	16	0.63	0.59	150
1550.0 x 300.0 x 172.7		21.38	1550	300	8	16	0.50	0.56	194
1600.0 x 320.0 x 205.98		7.53	1600	320	10	16	0.63	0.61	160
1700.0 x 320.0 x 187.14		15.87	1700	320	8	16	0.50	0.57	213
1700.0 x 380.0 x 240.84		4.62	1700	380	10	18	0.56	0.55	170
1850.0 x 280.0 x 204.1		3.54	1850	280	8	20	0.40	0.57	231
1850.0 x 320.0 x 225.61		13.19	1850	320	10	16	0.63	0.64	185
1850.0 x 340.0 x 222.94		1.65	1850	340	8	20	0.40	0.52	231
1950.0 x 320.0 x 263.6		0.11	1950	320	10	22	0.45	0.58	195

9.5.4 Recommended optimal set of welded I-sections

The recommended optimal set of welded I-sections was found to be the optimal set mentioned in Section 9.5.3 and listed in Table 9.22, being the most optimal set for beam and girder parameters which is smaller than 150 sections.

It should however be noted that if the optimal set size is not required to be smaller than 150 sections that the optimal sets mentioned in Sections 9.5.1 and 9.5.2 will combine to form the most optimal set of welded I-sections for beam and girder parameters.

9.6 Conclusion

It was found that the comparison methodology, provided in this chapter, is not sensitive to the different popularity weighting factors.

It was also found that the results produced by the optimisation methodology in terms of weight savings are not sensitive to the different popularity weighting factors, as the total mass difference between the A3 and B3 Optimal Set was found to be less than 1 % (refer to Section 9.3.3.1). In addition, about 78 % of the sections in the best B3 Optimal Set also occurred in the A3 Optimal Set with the same minimum rating of 90 (refer to Section 9.5.1). A change in weighting factors will change the sections present in the optimal set slightly, but these sections will have more or less the same dimensions and would not change the weight saving much.

The end result of the optimisation methodology can however be sensitive to the range of design parameters when they differ a lot, as mentioned in Section 9.3.3.1, because the optimisation methodology will only produce economical I-sections for the defined design parameters. The produced optimal set will thus not cover design parameters which are not defined. The defined design parameters is therefore very important in the process of obtaining an optimal set of welded I-sections.

This chapter provides a few optimal sets of welded I-section that can outperform the currently used hot-rolled I-sections. Of these optimal sets, the optimal sets obtained from Initial Set 2 (created with smaller dimensional increments) were found unjustified, as they only performed 1 % better than the optimal sets obtained from Initial Set 3 (practical initial set).

The most optimal set of welded I-sections for beam parameters can reduce the total mass (M) of the South African I-section by 18 % to 22 %, depending to which set of South African I-sections it is compared to (refer to Section 9.3.3).

The most optimal set of welded I-sections for girder parameters can reduce the total mass (M) of the South African I-section by 15 % to 19 %, depending to which set of South African I-sections it is compared to (refer to Section 9.4.2).

However, if these two optimal sets are combined in order to obtain the most optimal set for beams and girders, it becomes to large with 199 sections, which is 44 sections more than the set of I-sections (including hot-rolled I-sections and plate girders) currently used in South Africa.

The recommended optimal set of welded I-sections were produced by combining the two most optimal sets for beam and girder parameters and reducing them to 150 sections (refer to Section 9.5.3). In the process the mass reduction dropped to approximately 17 % for beam and girder parameters (refer to Section 9.5.3).

An optimal set of welded I-sections can thus reduce the total mass (M) of the South African I-section between 15 % and 22 %, regardless if it is used for beams or girders. The recommended optimal set of welded I-sections is presented in Table 9.22.

Chapter 10

Conclusions and recommendations

10.1 Achievement of objectives

The main objective of this research project was to produce an optimal set of standardised welded I-sections, ready for production by South African steel fabricators and to be documented in sources such as "The Red book" (SAISC, 2013). The study was, however, limited to the use of I-sections as beams and girders.

Firstly it was decided which parameters control the design of I-sections (span, load, lateral support conditions) and whether they act as beams or girders (beams that carry other beams). Design engineers and steel fabricators were then interviewed to assess how frequently certain values of these parameters occur in practice, to determine what the South African market actually uses.

The information from the interviews was used to define the "design space", based on the popularity (frequency of occurrence) of various values of each parameter, expressed as a weighting factor. For example a beam with both a span and a load that are seldom encountered in practice will, after multiplied by the weighting factors, have a very low weight. This process allowed the design of a "virtual project". The "virtual project" is a project that is, in effect, a scaled down version of all the steelwork projects in South Africa over a year period, thrown together as a single project.

An optimisation procedure was developed for the purpose of obtaining an optimal set of welded I-sections (refer to Chapter 5), starting with very large initial sets of sections with different dimensions and selecting those that are most popular over a region of the defined design space, while accounting for the practicality and capacity considerations at the same time. Sections that were too similar were also removed by the optimisation procedure.

A number of different optimal sets were produced by the optimisation procedure, based on different initial sets (Chapter 4) and different values of the design space (Chapter 7).

These optimal sets were compared with each other with the comparison methodology to assess the sensitivity to change in parameters and assumptions, to obtain the best optimal set of welded I-sections.

The best optimal set of welded I-sections were obtained for beam and girder conditions separately and were then combined to provide the optimal set of standardised welded I-sections, which can be used to replace the currently used hot-rolled I-sections and plate girders of South Africa.

10.2 Conclusions

The main findings of this research project is summarised as follows:

- This is, as far as can be established, the first time:
  - that anybody has tried to define an optimal set; and
  - that anybody has attempted to determine what steel I-sections are popular and for what they are actually used in South Africa.
- A preliminary optimal set of welded I-sections was obtained that can be used to replace the currently used South African hot-rolled I-sections and plate girders, for beam and girder conditions. This optimal set of welded I-sections can also be split into two separate sets, one to replace hot-rolled I-sections and one to replace plate girders.
- The optimal set of standardised welded I-sections was found to be between 17 % and 19 % lighter than the currently used South African I-sections (hot-rolled I-sections and plate girders) in terms of total mass over a virtual project (Design space B and C). If the South African IPE's are used in combination with the optimal set, the total mass of the virtual project reduce by as much as 21 %.
- The optimal set of standardised welded I-sections was also found to be between 18 % and 23 % lighter than the currently used South African hot-rolled I-sections (on its own) in terms of total mass over a virtual project (Design space B and C) and approximately 16 % lighter than the currently used plate girders in South Africa.
- Steel Services found that welded I-sections, with similar dimensions to South African hot-rolled I-sections, cost between 20 % and 40 % more than the corresponding hot-rolled I-sections. Should the optimal set of standardised welded I-sections be used instead, the cost of welded I-sections can be reduced up to 23 %, as the mentioned weight reductions will convert to cost reductions. It can thus be economically viable to replace the currently used hot-rolled I-sections in South Africa with welded I-sections, as the cost of hot-rolled I-sections are higher in South Africa than in other countries, because medium to heavy hot-rolled I-sections are not produced in South Africa anymore, but imported. With import cost and time delays, welded I-sections are already preferred by some steel fabricators above hot-rolled I-sections in South Africa.

Concerning the available optimisation methods, the following was noted:

- Minimum cost optimisation was found to be complex, as a lot of information is needed to produce a suitable cost function. Mela and Heinisuo (2014) also found that the results of minimum cost and weight optimisation do not differ much from each other for welded I-sections, including homogeneous and hybrid I-sections. Minimum cost optimisation is also not necessarily more accurate than weight optimisation, as companies and people differ from each other.

The following can be concluded regarding the optimisation and comparison methodology:

- Although the optimisation methodology presented in this thesis was limited to beams and girders in South Africa, the main outline of the optimisation methodology can be used to obtain an optimal set of any structural element in any country. This is also the case for the comparison methodology.
- The optimisation methodology was not really sensitive to different popularity weighting factors, but are quite sensitive to the size of the initial set of I-sections and the variation in minimum rating.
- The results produced by the optimisation methodology in terms of weight savings are not sensitive to the different popularity weighting factors, but could be sensitive to the design parameter ranges when they differ a lot. The optimisation methodology will only produce economical I-sections for the defined design parameters. The produced optimal set will thus not cover design parameters which are not defined. The defined design parameters are therefore very important in the process of obtaining an optimal set of welded I-sections.
- The optimisation methodology assumed that the popularities of different design parameters do not have an influence of each other. For example, the span of a beam does not have an influence on the load per meter applied to the beam. This is not necessarily the case in practice, but this could not be accounted for in this research project, as only limited information was available, forcing the popularity of different design parameters to be determined separately and applied as such.

Concerning the field work and popularity weighting factors, the following was noted:

- The design assumptions varied from one engineer to the next. For example, Participant 1, in line with the Red Book (SAISC, 2013), assumes that grating does not provide any lateral support, while Participant 2 assumes that grating does provide lateral support.
- It was found from the fieldwork that engineers normally design their beams and girders as pinned. This supports the decision to limit this study to simply supported beams and girders.
- The information used to determine the popularity weighting factors was limited to a survey amongst a limited number of engineers who design different projects and do not necessarily represent the entire population of engineers of South Africa. A comprehensive survey of all the applications of I-sections for all types of structures is required to produce the final optimal set of I-sections for practice, but a comprehensive survey would have taken too long and been too expensive for this research project. For this reason the research project focused more on the development of the optimisation methodology to obtain an optimal

set of welded I-sections than to produce a final optimal set of I-sections for practice.

- The optimisation approach accepted what is done in practice as popular, instead of using the design parameters which are thought to be popular in practice, making the approach more practical and relevant.

Concerning the optimal sets of welded I-sections, the following was noted:

- The top three I-sections in the optimal sets obtained from Initial Set 1, the initial set created by approximating hot-rolled I-sections available globally, corresponded to the currently popular hot-rolled I-sections of South Africa, namely the IPE 200, UB 254 x 146 x 31 and UB 305 x 102 x 25. This demonstrates that the optimisation methodology does single out the good sections.

- The web to flange thickness ratio (t_w/t_f) of the optimal sets were never smaller than 0.28, under beam and girder conditions. It can thus be concluded that practical I-sections will never have a web to flange thickness ratio of less than 0.28, as the minimum web to flange thickness ratio used in the creation of the initial set of welded I-sections was 0.17.

- The web to total area ratio (A_w/A) of the optimal welded I-sections were not always in the region of 0.63, which corresponds to the practical web to total area ratios derived by Schilling (1974). It was normally between 0.39 and 0.62, which corresponds to the region defined by Schilling (1974) for I-sections with an optimal elastic sections modulus. This means that the derivations of Schilling (1974) is correct, but do not always work in reality were the sections can not be fabricated to specific area ratios.

10.3 Recommendations

Aligned with objectives and conclusions of this research project, the following is recommended:

- A comprehensive survey of all the applications of I-sections for all types of structures can be undertaken, in order to determine and confirm the design parameter ranges and popularity weighting factors of the different design parameters more accurately, compared to the popularity weighting factors and design parameter ranges mentioned in this thesis. This will lead to the identification of a set of welded I-sections that are more attuned to the actual needs of the country.

- As this research project only accounted for the popularity of design parameters separately, further study into determining the popularity of the combination of design parameters is recommended.

- As this thesis only focused on simply supported beams and girders, further study based on the optimisation methodology on the use of I-sections as columns is recommended. This study will make it possible to produce an optimal set of columns, which can then be combined with the optimal sets in this research project in order to produce an optimal set of welded I-sections that covers the full I-section spectrum.

- The I-sections in this research project were optimised according to weight and not cost. Although cost of welded I-sections is closely linked to their weight, full cost optimisation could give slightly different results. It is therefore recommended to optimise the welded I-sections according to cost with the use of the optimisation methodology to confirm the optimal set of standardised welded I-sections provided in this research project.

10.4 Concluding statement

This research project provides a preliminary optimal set of standardised welded I-sections for beams and girders. It also demonstrates that it can be economically viable to replace the currently used hot-rolled I-sections in South Africa with welded I-sections. This research project furthermore provides the basis for future research in the development of an optimal set of standardised welded I-sections, in order to include columns and a economical evaluation of the production of welded I-sections.

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Appendix A

South African I-sections

Table A.1: Dimensions of South African hot-rolled I-sections (SAISC, 2013)

Section Name $h \times b \times m$	m kg/m	h mm	b mm	t_w mm	t_f mm	$r1$ mm	h_w mm
IPE sections							
IPE-AA 100	6.72	97.60	55.00	3.60	4.50	7.00	74.60
IPE 100	8.10	100.0	55.00	4.10	5.70	7.00	74.60
IPE-AA 120	8.36	117.0	64.00	3.80	4.80	7.00	93.40
IPE 120	10.40	120.0	64.00	4.40	6.30	7.00	93.40
IPE-AA 140	10.10	136.6	73.00	3.80	5.20	7.00	112.0
IPE 140	12.90	140.0	73.00	4.70	6.90	7.00	112.0
IPE-AA 160	12.30	156.4	82.00	4.00	5.60	9.00	127.0
IPE 160	15.80	160.0	82.00	5.00	7.40	9.00	127.0
IPE-AA 180	14.90	176.4	91.00	4.30	6.20	9.00	146.0
IPE 180	18.80	180.0	91.00	5.30	8.00	9.00	146.0
IPE-AA 200	18.00	196.4	100.0	4.50	6.70	12.00	159.0
IPE 200	22.40	200.0	100.0	5.60	8.50	12.00	159.0
Universal beams (UB)							
203x133x25	25.10	203.2	133.4	5.70	7.80	7.60	172.4
203x133x30	30.00	206.8	133.8	6.40	9.60	7.60	172.4
254x146x31	31.10	251.5	146.1	6.00	8.60	7.60	219.1
254x146x37	37.00	256.0	146.4	6.30	10.90	7.60	219.0
254x146x43	43.00	259.6	147.3	7.20	12.70	7.60	219.0
305x102x25	24.80	304.8	101.6	5.80	7.00	7.60	275.6
305x102x28	28.20	308.9	101.9	6.00	8.80	7.60	276.1
305x102x33	32.80	312.7	102.4	6.60	10.80	7.60	275.9
305x165x40	40.30	303.8	165.1	6.10	10.20	8.90	265.6

Table A.1 (continued)

Section Name $h \times b \times m$	m **kg/m**	h **mm**	b **mm**	t_w **mm**	t_f **mm**	$r1$ **mm**	h_w **mm**
305x165x46	46.10	307.1	165.7	6.70	11.80	8.90	265.7
305x165x54	54.00	310.9	166.8	7.70	13.70	8.90	265.7
356x171x45	45.00	352.0	171.0	6.90	9.70	10.20	312.2
356x171x51	51.00	355.6	171.5	7.30	11.50	10.20	312.2
356x171x57	57.00	358.6	172.1	8.00	13.00	10.20	312.2
356x171x67	67.10	364.0	173.2	9.10	15.70	10.20	312.2
406x140x39	39.00	397.3	141.8	6.30	8.60	10.20	359.7
406x140x46	46.00	402.3	142.4	6.90	11.20	10.20	359.5
406x178x54	54.10	402.6	177.6	7.60	10.90	10.20	360.4
406x178x60	60.10	406.4	177.8	7.80	12.80	10.20	360.4
406x178x67	67.10	409.4	178.8	8.80	14.30	10.20	360.4
406x178x74	74.20	412.8	179.7	9.70	16.00	10.20	360.4
457x191x67	67.10	453.6	189.9	8.50	12.70	10.20	407.8
457x191x74	74.30	457.2	190.5	9.10	14.50	10.20	407.8
457x191x82	82.00	460.2	191.3	9.90	16.00	10.20	407.8
457x191x89	89.30	463.6	192.0	10.60	17.70	10.20	407.8
457x191x98	98.30	467.6	192.8	11.40	19.60	10.20	408.0
533x210x82	82.20	528.3	208.7	9.60	13.20	12.70	476.5
533x210x92	92.10	533.1	209.3	10.20	15.60	12.70	476.5
533x210x101	101.0	536.7	210.1	10.90	17.40	12.70	476.5
533x210x109	109.0	539.5	210.7	11.60	18.80	12.70	476.5
533x210x122	122.0	544.6	211.9	12.80	21.30	12.70	476.6
Universal columns (UC)							
152x152x23	23.00	152.4	152.4	6.10	6.80	7.60	123.6
152x152x30	30.00	157.5	152.9	6.60	9.40	7.60	123.5
152x152x37	37.00	161.8	154.4	8.10	11.50	7.60	123.6
203x203x46	46.10	203.2	203.2	7.30	11.00	10.20	160.8
203x203x52	52.00	206.2	203.9	8.00	12.50	10.20	160.8
203x203x60	60.00	209.6	205.2	9.30	14.20	10.20	160.8
203x203x71	71.00	215.9	206.2	10.30	17.30	10.20	160.9
203x203x86	86.10	222.3	208.8	13.00	20.50	10.20	160.9
254x254x73	73.10	254.2	254.0	8.60	14.20	12.70	200.4

Table A.1 (continued)

Section Name $h \times b \times m$	m kg/m	h mm	b mm	t_w mm	t_f mm	$r1$ mm	h_w mm
254x254x89	88.90	260.4	255.9	10.50	17.30	12.70	200.4
254x254x107	107.0	266.7	258.3	13.00	20.50	12.70	200.3
254x254x132	132.0	276.4	261.0	15.60	25.10	12.70	200.8
254x254x167	167.0	289.1	264.5	19.20	31.70	12.70	200.3
305x305x97	96.90	307.8	304.8	9.90	15.40	15.20	246.6
305x305x118	118.0	314.5	306.8	11.90	18.70	15.20	246.7
305x305x137	137.0	320.5	308.7	13.80	21.70	15.20	246.7
305x305x158	158.0	327.2	310.6	15.70	25.00	15.20	246.8
305x305x198	198.0	339.9	314.1	19.20	31.40	15.20	246.7
305x305x198	240.0	352.6	317.9	23.00	37.70	15.20	246.8
Taper flange I-sections (J) $\beta = 98°$							
Section Name $h \times b \times m$	m kg/m	h mm	b mm	t_w mm	t_f mm	$r1$ mm	$r2$ mm
203x152x52	52.30	203.0	152.0	8.90	16.50	15.50	7.60

Table A.2: Dimensions of South African plate girders (SAISC, 2013)

Section Name $h \times b \times (t_w, t_f)$	m kg/m	h mm	b mm	t_w mm	t_f mm	h_w^* mm
704 x 200 (8W, 12F)	80.38	704.0	200.0	8.00	12.00	680.0
712 x 200 (8W, 16F)	92.94	712.0	200.0	8.00	16.00	680.0
720 x 200 (8W, 20F)	105.5	720.0	200.0	8.00	20.00	680.0
704 x 250 (8W, 12F)	89.80	704.0	250.0	8.00	12.00	680.0
712 x 250 (8W, 16F)	105.5	712.0	250.0	8.00	16.00	680.0
720 x 250 (8W, 20F)	121.2	720.0	250.0	8.00	20.00	680.0
712 x 300 (8W, 16F)	118.1	712.0	300.0	8.00	16.00	680.0
720 x 300 (8W, 20F)	136.9	720.0	300.0	8.00	20.00	680.0
812 x 200 (8W, 16F)	99.22	812.0	200.0	8.00	16.00	780.0
820 x 200 (8W, 20F)	111.8	820.0	200.0	8.00	20.00	780.0
812 x 250 (8W, 16F)	111.8	812.0	250.0	8.00	16.00	780.0
820 x 250 (8W, 20F)	127.5	820.0	250.0	8.00	20.00	780.0
830 x 250 (8W, 25F)	147.1	830.0	250.0	8.00	25.00	780.0
812 x 300 (8W, 16F)	124.3	812.0	300.0	8.00	16.00	780.0

Table A.2 (continued)

Section Name $h \times b \times (t_w, t_f)$	m kg/m	h mm	b mm	t_w mm	t_f mm	h_w^* mm
830 x 250 (8W, 25F)	147.1	830.0	250.0	8.00	25.00	780.0
812 x 300 (8W, 16F)	124.3	812.0	300.0	8.00	16.00	780.0
820 x 300 (8W, 20F)	143.2	820.0	300.0	8.00	20.00	780.0
830 x 300 (8W, 25F)	166.7	830.0	300.0	8.00	25.00	780.0
904 x 200 (8W, 12F)	92.94	904.0	200.0	8.00	12.00	880.0
912 x 200 (8W, 16F)	105.5	912.0	200.0	8.00	16.00	880.0
920 x 200 (8W, 20F)	118.1	920.0	200.0	8.00	20.00	880.0
904 x 250 (8W, 12F)	102.4	904.0	250.0	8.00	12.00	880.0
912 x 250 (8W, 16F)	118.1	912.0	250.0	8.00	16.00	880.0
920 x 250 (8W, 20F)	133.8	920.0	250.0	8.00	20.00	880.0
930 x 250 (8W, 25F)	153.4	930.0	250.0	8.00	25.00	880.0
904 x 300 (8W, 12F)	111.8	904.0	300.0	8.00	12.00	880.0
912 x 300 (8W, 16F)	130.6	912.0	300.0	8.00	16.00	880.0
920 x 300 (8W, 20F)	149.5	920.0	300.0	8.00	20.00	880.0
930 x 300 (8W, 25F)	173.0	930.0	300.0	8.00	25.00	880.0
1004 x 250 (8W, 12F)	108.6	1004	250.0	8.00	12.00	980.0
1012 x 250 (8W, 16F)	124.3	1012	250.0	8.00	16.00	980.0
1020 x 250 (8W, 20F)	140.0	1020	250.0	8.00	20.00	980.0
1030 x 250 (8W, 25F)	159.7	1030	250.0	8.00	25.00	980.0
1012 x 300 (8W, 16F)	136.9	1012	300.0	8.00	16.00	980.0
1020 x 300 (8W, 20F)	155.7	1020	300.0	8.00	20.00	980.0
1030 x 300 (8W, 25F)	179.3	1030	300.0	8.00	25.00	980.0
1040 x 300 (8W, 30F)	202.8	1040	300.0	8.00	30.00	980.0
1020 x 400 (8W, 20F)	187.1	1020	400.0	8.00	20.00	980.0
1030 x 400 (8W, 25F)	218.5	1030	400.0	8.00	25.00	980.0
1040 x 400 (8W, 30F)	249.9	1040	400.0	8.00	30.00	980.0
1204 x 250 (8W, 12F)	121.2	1204	250.0	8.00	12.00	1180
1212 x 250 (8W, 16F)	136.9	1212	250.0	8.00	16.00	1180
1220 x 250 (8W, 20F)	152.6	1220	250.0	8.00	20.00	1180
1230 x 250 (8W, 25F)	172.2	1230	250.0	8.00	25.00	1180
1212 x 300 (8W, 16F)	149.5	1212	300.0	8.00	16.00	1180
1220 x 300 (8W, 20F)	168.3	1220	300.0	8.00	20.00	1180

Table A.2 (continued)

Section Name $h \times b \times (t_w, t_f)$	m **kg/m**	h **mm**	b **mm**	t_w **mm**	t_f **mm**	h_w^* **mm**
1230 x 250 (8W, 25F)	172.2	1230	250.0	8.00	25.00	1180
1212 x 300 (8W, 16F)	149.5	1212	300.0	8.00	16.00	1180
1220 x 300 (8W, 20F)	168.3	1220	300.0	8.00	20.00	1180
1230 x 300 (8W, 25F)	191.9	1230	300.0	8.00	25.00	1180
1240 x 300 (8W, 30F)	215.4	1240	300.0	8.00	30.00	1180
1220 x 400 (8W, 20F)	199.7	1220	400.0	8.00	20.00	1180
1230 x 400 (8W, 25F)	231.1	1230	400.0	8.00	25.00	1180
1240 x 400 (8W, 30F)	262.5	1240	400.0	8.00	30.00	1180
1402 x 250 (10W, 16F)	170.3	1402	250.0	10.00	16.00	1370
1410 x 250 (10W, 20F)	186.0	1410	250.0	10.00	20.00	1370
1420 x 250 (10W, 25F)	205.7	1420	250.0	10.00	25.00	1370
1402 x 300 (10W, 16F)	182.9	1402	300.0	10.00	16.00	1370
1410 x 300 (10W, 20F)	201.7	1410	300.0	10.00	20.00	1370
1420 x 300 (10W, 25F)	225.3	1420	300.0	10.00	25.00	1370
1430 x 300 (10W, 30F)	248.8	1430	300.0	10.00	30.00	1370
1410 x 400 (10W, 20F)	233.1	1410	400.0	10.00	20.00	1370
1420 x 400 (10W, 25F)	264.5	1420	400.0	10.00	25.00	1370
1430 x 400 (10W, 30F)	295.9	1430	400.0	10.00	30.00	1370
1440 x 400 (10W, 35F)	327.3	1440	400.0	10.00	35.00	1370
1602 x 300 (12W, 16F)	223.3	1602	300.0	12.00	16.00	1570
1610 x 300 (12W, 20F)	242.1	1610	300.0	12.00	20.00	1570
1620 x 300 (12W, 25F)	265.6	1620	300.0	12.00	25.00	1570
1630 x 300 (12W, 30F)	289.2	1630	300.0	12.00	30.00	1570
1610 x 400 (12W, 20F)	273.5	1610	400.0	12.00	20.00	1570
1620 x 400 (12W, 25F)	304.9	1620	400.0	12.00	25.00	1570
1630 x 400 (12W, 30F)	336.3	1630	400.0	12.00	30.00	1570
1640 x 400 (12W, 35F)	367.7	1640	400.0	12.00	35.00	1570
1620 x 500 (12W, 25F)	344.1	1620	500.0	12.00	25.00	1570
1630 x 500 (12W, 30F)	383.4	1630	500.0	12.00	30.00	1570
1640 x 500 (12W, 35F)	422.6	1640	500.0	12.00	35.00	1570
1820 x 300 (12W, 25F)	284.5	1820	300.0	12.00	25.00	1770
1830 x 300 (12W, 30F)	308.0	1830	300.0	12.00	30.00	1770

Table A.2 (continued)

Section Name $h \times b \times (t_w, t_f)$	m **kg/m**	h **mm**	b **mm**	t_w **mm**	t_f **mm**	h_w^* **mm**
1810 x 400 (12W, 20F)	292.3	1810	400.0	12.00	20.00	1770
1820 x 400 (12W, 25F)	323.7	1820	400.0	12.00	25.00	1770
1830 x 400 (12W, 30F)	355.1	1830	400.0	12.00	30.00	1770
1840 x 400 (12W, 35F)	386.5	1840	400.0	12.00	35.00	1770
1820 x 500 (12W, 25F)	363.0	1820	500.0	12.00	25.00	1770
1830 x 500 (12W, 30F)	402.2	1830	500.0	12.00	30.00	1770
1840 x 500 (12W, 35F)	441.5	1840	500.0	12.00	35.00	1770
1850 x 500 (12W, 40F)	480.7	1850	500.0	12.00	40.00	1770
2120 x 300 (14W, 25F)	345.2	2120	300.0	14.00	25.00	2070
2130 x 300 (14W, 30F)	368.8	2130	300.0	14.00	30.00	2070
2140 x 300 (14W, 35F)	392.3	2140	300.0	14.00	35.00	2070
2110 x 400 (14W, 20F)	353.1	2110	400.0	14.00	20.00	2070
2120 x 400 (14W, 25F)	384.5	2120	400.0	14.00	25.00	2070
2130 x 400 (14W, 30F)	415.9	2130	400.0	14.00	30.00	2070
2140 x 400 (14W, 35F)	447.3	2140	400.0	14.00	35.00	2070
2120 x 500 (14W, 25F)	423.7	2120	500.0	14.00	25.00	2070
2130 x 500 (14W, 30F)	463.0	2130	500.0	14.00	30.00	2070
2140 x 500 (14W, 35F)	502.2	2140	500.0	14.00	35.00	2070
2150 x 500 (14W, 40F)	541.5	2150	500.0	14.00	40.00	2070

Appendix B

Interview with steel fabricator

The objective of this interview was to determine the practical considerations that need to be met to create I-sections that are practical, fabricate-able and economical.

The practical considerations could only be obtained by interviewing a steel fabricator, because literature or data from practice don not account for the experience of a steel fabricator.

The CEO of Union Steel was interviewed, because Union Steel is one of the leading steel fabricators in the Western Cape and their staff members have extensive experience in the fabrication of welded I-sections. The interview is regarded sufficient for the purposes of this study, as the practical considerations to be considered when fabricating welded I-sections are the same for all steel fabricators.

B.1 Participant information

Company:	Union Structural Engineering Works (Pty) Ltd. (Union Steel)
Position:	CEO of Union Steel
Academic Qualifications:	B.Sc. (Eng)
Experience:	24 years' experience in the steel industry

B.2 Questions with Answers

This section contains all the information obtained from the interview with the above mentioned participant, conducted by the investigator (Mr N. Tredoux) and his supervisor (Dr H. De Clercq). The questions and corresponding answers were as follows:

1 What are the standard plate sizes Union Steel uses to construct welded I-sections?

Union steel always keep standard plate sizes of 2.4 x 10 m in stock.

The standard plate width Union Steel normally uses is 2.4 m.

Hot-rolled sheets can also be used as an alternative to plates, but are only available in widths of up to 2 m and thicknesses of up to 12 mm.

Flat bars are good to use for flanges. Union Steel usually uses flat bars with widths of 130 mm, 150 mm, 180 mm and 200 mm. Flat bars of 250 mm can also be used for flanges of welded I-sections. Flat bars with widths of 110 mm are not usually used, as they are hard to find.

2 How much does cutting defects influence the flanges and webs of welded I-sections?

At Union Steel all plates are trimmed by 20 mm on each side of the plate before flanges and webs are cut. This removes any defects on the plate edges caused by transport, handling, etc.

When flanges and webs are cut out of plates, it is done simultaneously on both sides of the plate. This is done to minimise plate distortion.

3 What is the quality of steel mostly used by you to construct welded I-sections and does it influence your welding time and the welds you use?

The plate size has a bigger influence on the type of weld that can be applied than the quality of steel, when fabricating welded I-sections.

Fillet welding by hand is usually used for thinner plates, because it does not expose the welded plate to high temperature, as is the case with submerged arc welding.

Submerged arc welding is used on thicker plates and is usually done by automated or semi-automated machinery. Submerged arc welding is also faster than fillet welding, but can only be used on thick plates, as it generates intense amounts of heat.

The welding production rates for 6 mm fillet welds are at a rate of $\pm$ 4 m/h and submerged arc welding are at a rate of $\pm$ 40 m/3h.

4 What are the minimum and maximum flange sizes you would use to construct welded I-sections?

The minimum flange width that could be used is 100 mm, because it becomes impossible to drill or punch holes in flanges less than 100 mm wide.

5 What type of corrosion protection do you use most on your welded I-sections?

Union Steel uses a variety of protection techniques to protect welded I-sections from corrosion, but galvanizing is a big "NO-NO" for welded I-sections. Welded I-sections distort a great deal during the galvanizing process.

6 Is there any handleability boundary conditions which should be taken into account in the design of an optimal set of welded I-sections?

For weldability purposes, the webs and flanges should not have a thicknesses less than 5 mm. A 4 mm plate can still be welded, but it becomes difficult.

Usually beams are handled in lengths of up to 13 m. Beam lengths are not really a problem for steel fabricators, as the production line at Union Steel can handle plates of up to 24 m. Steel handling is the main job of steel fabricators.

7 What are the most used I-sections?

- Beams: 254 x 146 x 31 I-section
- Columns: 356 x 171 x 45 I-section

8 How much cheaper are plates per ton than hot-rolled I-sections?

Plates are about R 1000 to R 2000 per ton cheaper than hot-rolled I-sections.

9 General information

400-500 MPa steel is not available in South Africa.

Appendix C

Interviews with engineers from practice

The objective of the interviews with engineers from practice was to obtain sufficient information to identify the popular design parameters (the design parameters which are used frequently) and the frequency of occurrence of the different design parameters.

The frequently occurring design parameters will be used to identify the popular design space. The popular design space is defined by the support conditions (laterally supported or unsupported), span, effective length and load (distributed or point loads).

Four participants from four different companies were interviewed. The information obtained from the interviews is presented in the following sections.

C.1 Participant 1

C.1.1 Participant information

Position at company:	Director
Academic Qualifications:	BEng, MEng and PrEng
Experience:	23 years of experience in steel and concrete structures

C.1.2 Questions with Answers

This section contains all the information obtained from the interview with the above mentioned participant, conducted by the investigator (Mr N. Tredoux). The questions and corresponding answers were as follows:

C.1.2.1 General engineering questions

1 What are the typical steel structures you normally design and how many of them per year?

The participant is involved in approximately 400 projects per year of which approximately 300 are steel structures and 50 large structures. The participant is mostly responsible for the large structures, which are mostly classified as steel structures.

2 Which I-sections do you use most often and why?

For Beams (secondary beams): I 203 x 133 x 25

The beams are normally laterally unsupported, as concrete floors on metal deck with shear studs are rarely used.

The beams normally support grating, which is assumed not to give any lateral support to the beams. This assumption is made because of the type of grating that is used in the structures and because some clients require that the mentioned assumption must be used.

For Girders (primary beams): I 305 x 165 x 46

Welded plate girders are not used often. Plate girders are used about 10 % of the time. The reason for this is that welded plate girders are normally more expensive. When it needs to be used because of the strength constraints of hot-rolled I-sections, the plate girders in the Red Book (SAISC, 2013) is not used, because of limited space (too deep). Normally the design starts with a plate girder out of the Red Book (SAISC, 2013) and then modifies it to fit into the available space (depth), while still meeting the strength requirements.

For Columns: H 152 x 152 x 30 and H 203 x 203 x 46

These H-sections are used as columns for its higher weak-axis bending resistance compared to I-sections. These sections are also popular for practical connection design.

C.1.2.2 Engineering questions applicable to beams (secondary beams)

1 How often do you use fixed or simply supported beams (percentages)?

- Percentage pinned beams: 80 %
- Percentage fixed beams: 20 %

2 What are the typical floors in the steel structures you design?

Grating is normally used, as the projects are normally industrial structures. Grating is also easier to construct when compared to Vastrap. Grating is clipped to the beams, while Vastrap needs to be welded. This makes Vastrap also more expensive than grating.

Concrete floors are not normally used, due to construction time, etc. When concrete floors are used, concrete floors with a metal deck are mostly used, with no composite action and with a 50 mm screed layer (only in office buildings). Hollow core floors are not used, as they are not popular in industrial structures because of their low vibration resistance and stiffness in general. They can also not be used to transfer forces like solid concrete floors (like floor bracing) due to the weaker membrane stiffness. Hollow core floors are more popular in houses. Concrete floors normally range from 170 mm to 255 mm in houses (which correspond to 2 to 3 brick layers).

Table C.1 provides an indication of how frequently the different floor types are used or how popular they are in the structural projects of the participant.

Table C.1: Popularity of typical floor structures according to Participant 1

Structure:	Industrial structures	
Floor type	**Thickness range (mm)**	**Percentage**
Grating	30	90 %
Vastrap	4.5 to 6	9 %
Concrete on metal deck (composite)	200	1 %

3 What are the typical imposed loads on the floors, roofs and beams (if applicable) of the structures you design?

The following table provides the typical imposed loads used by the participant.

Table C.2: Typical imposed loads according to Participant 1

Part of structure	**Live load (kPa)**	**Other imposed loads (kPa)**
Industrial floors	5	1 (services)
Industrial roofs	1	0.5 (services)

4 What are the typical wind loads on the roofs of the structures you design?

The wind loads are different for each structure, but normally they range between 0.3 kPa to 1 kPa. The wind load on the front of the structure are normally approximately 0.8 kPa, at the back in the range of 0.3 to 0.4 kPa and approximately 1 kPa on the side.

5 What are the typical permanent loads of the structures you design?

The permanent loads normally include the self-weight of the material, roofs and floors. When concrete floors are used in office buildings, the 50 mm screed is also accounted for.

6 Are the I-beams normally laterally supported or unsupported in the structures you design (percentage)?

- Percentage laterally supported beams: 5 %

  Beams supporting Vastrap or concrete floors, constructed as part of a composite beam, are the only beams that are considered to be laterally supported.

- Percentage laterally unsupported beams: 95 %

  Beams supporting grating and non-composite concrete floors are all considered to be laterally unsupported.

7 What are the typical spans of the I-beams in the structures you design and how often do they occur (percentage)?

The beam spans can range between 2 m and 4 m. Normally a maximum span of 3 m is used.

No percentages can be supplied on how much certain spans occur.

C.1.2.3 Engineering questions applicable to girders (primary beams)

1 How often do you use welded plate girders?

Plate girders are not used often, as they are more expensive than rolled sections.

2 How often do you use fixed or simply supported girders (percentages)?

The same percentages as for beams.

- Percentage pinned girders: 80 %
- Percentage fixed girders: 20 %

3 Are the applied loads normally distributed or point loads on the plate girders used and what are they typically?

The applied loads are normally point loads between 30 to 70 kN, with point loads between 30 and 50 kN occurring the most.

4 Are the girders normally laterally supported or unsupported in the structures you design (percentage)?

The girders are always laterally unsupported and normally braced every 3 m. Thus 100 % of girders are laterally unsupported.

5 What are the typical spans of the girders in the structures you design and how often do they occur (percentage)?

Girder spans of 5 m are usually used, but they can span up to 20 m in some structures.

No percentages can be supplied on how much certain spans occur.

6 What are the typical effective lengths (beam spacing) of the girders in the structures you design and how often do they occur (percentage)?

The beam spacing and effective length are not always the same. The spacing of the beams are normally 1.5 m or smaller, because of the span limits of grating. The effective lengths are normally between 3 m (60 % of the time) and 5 m (40 % of the time).

C.1.2.4 Engineering questions applicable to beams and girders

1 Is there any other information you feel I should know to obtain an optimal set of welded I-sections?

Fabrication cost and connection design should be accounted for in the optimisation process of obtaining an optimal set of welded I-sections. This could be done in this thesis or in future research.

Fabrication cost should be assessed to make the set more optimal. Connection design is also important for welded I-sections, because they fail mostly in bearing at the connections due to the thin webs of welded I-sections.

The bearing and block shear of M20 bolts can be accounted for in the connection design. M20 bolts are good to consider, because they are normally used to connect beams to columns.

C.2 Participant 2

C.2.1 Participant information

Position at company:	Design Engineer
Academic Qualifications:	B.Sc. (Eng), M.Sc. (Eng), GDE and PrEng
Experience:	7 years of experience in steel and concrete structures

C.2.2 Questions with Answers

This section contains all the information obtained from the interview with the above mentioned participant, conducted by the investigator (Mr N. Tredoux). The questions and corresponding answers were as follows:

C.2.2.1 General engineering questions

1 What are the typical steel structures you normally design and how many of them per year?

The participant has approximately 30 to 40 projects per year. The structures vary between industrial structures, warehouses, shopping centres, office blocks, etc. and they range from R 1 million to R 1 billion in cost.

The participant is involved in the management and design of these projects.

2 Which I-sections do you use most often and why?

For Beams (secondary beams):

Different groups of I-sections are used for composite beams and platforms. The I-sections are grouped as follows:

– Composite beams: IPE 160
  UB 305 x 102 x 25
  UB 406 x 140 x 39
  UB 533 x 210 x 82

– Platforms: IPE 160
  $\mathrm{IPE}_{\mathrm{AA}}$ 160
  IPE 200
  UB 305 x 102 x 25
  UB 406 x 140 x 39
  UB 457 x 191 x 67

It is assumed that the grating provides lateral support. There is also a paper available from the University of the Witwatersrand which proves that grating provides lateral support to beams.

The British universal beams (UB) manufactured in South Africa are normally used. They are slightly different from the British universal beams (UB) manufactured overseas.

For Girders (primary beams): UB 406 x 140 x 39
UB 457 x 191 x 67
UB 533 x 210 x 82

Plate girders are used as needed. When plate girders are used, they are obtained from the Red Book (SAISC, 2013), as the structure do not have depth constraints specified for the girders.

For Columns: UC 152 x 23
UC 203 x 46
UC 254 x 73
UC 305 x 97
UB 356 x 171 x 45

The UB 356 x 171 x 45 is normally used for the main columns of the structures.

C.2.2.2 Engineering questions applicable to beams (secondary beams)

1 How often do you use fixed or simply supported beams (percentages)?

- Percentage pinned beams: 90 - 95 %
- Percentage fixed beams: 5 - 10 %

2 What are the typical floors in the steel structures you design?

As mentioned above, the participant designs various different structures.

The warehouses do not contain much suspended floors supported by beams; only the offices in the warehouses are sometimes build on floors supported by beams. The office buildings and industrial buildings, on the other hand, contain more beams and suspended floors.

The industrial buildings are categorised into two groups: (Group 1) industrial structures with more work areas and (Group 2) industrial structures with more machinery (Refer to Table C.3).

The industrial structures are normally a combination of the two groups, but the different groups have different requirements when it comes to floors. The work areas are normally constructed with more concrete floors on metal deck due to vibration constraints, while the floors in the structures with machinery do not have such constraints. The floors of Group 2 are normally grating or Vastrap, depending on the structure and client requirements.

Normally concrete floors with shear studs are used in industrial structures, with the beams behaving as composite beams. The reason for this being that the floors need to withstand large loads. Concrete floors with a metal deck and without shear studs are only used when there are plans to expand the structure in the future, as they are easier to change when another type of floor is needed.

Office buildings are normally constructed with normal concrete floors (in-situ concrete), but when needed composite floors (concrete floors with shear studs) are also used (refer to Table C.4).

The normal concrete floors are normally constructed with permanent shuttering on I-sections, but it is possible to construct them without shuttering. Bond-dek and Bond-lok are some of the permanent shuttering methods that can be used.

Table C.3: Popularity of typical industrial floor structures according to Participant 2

Structures:	Industrial structures (Group 1)		Industrial structures (Group 2)	
Floor type	**Thickness range (mm)**	**Percentage**	**Thickness range (mm)**	**Percentage**
Grating	30 x 4.5 40 x 3	40 %	30 x 4.5 40 x 3	50 %
Vastrap			6 x 8	50 %
Concrete on metal deck (composite)	170 - 200 270 (30 kPa)	60 %		

Table C.4: Popularity of typical floor structures in office buildings according to Participant 2

Floor type	Thickness range (mm)	Percentage
Normal concrete	140 - 170	90 - 95 %
Concrete on metal deck (composite)	140 - 170	5 -10 %

3 What are the typical imposed loads on the floors, roofs and beams (if applicable) of the structures you design?

The imposed loads normally include all the imposed loads in one, from the live loads specified by the loading codes to the services in the structure. The following table provides the typical imposed loads used.

Table C.5: Typical imposed loads according to Participant 2

Part of structure	Imposed loads (kPa)
Industrial platforms (floors)	5
Industrial storage areas (floors)	10
Industrial rebuild areas (floors)	20
Industrial roofs	5
Office building floors	3

As shown in Table C.5, the imposed loads used for rebuilt areas and roofs are much more than what is specified in the South African loading codes (SABS, 2011b). Rebuilt areas are the areas that can possibly be used for other purposes in future. For this reason 20 kPa is always used, because of the uncertainty of the loads which will be applied to the floors. Industrial roofs are also designed to withstand higher loads, because they are sometimes used for storage space even when they are not designed for it.

4 What are the typical wind loads on the roofs of the structures you design?

The wind loads are different for each structure, but they normally range between 0.7 kPa and 1 kPa unfactored.

5 What are the typical permanent loads of the structures you design?

The permanent loads are normally just the self-weight of the material, roofs and floors. When concrete floors are used in office buildings, 50 mm screed or tiles is also accounted for. When concrete floors are used in industrial buildings, screed is normally not used.

6 Are the I-beams normally laterally supported or unsupported in the structures

you design (percentage)?

- Percentage laterally supported beams: 70 %
- Percentage laterally unsupported beams: 30 %

7 What are the typical spans of the I-beams in the structures you design and how often do they occur (percentage)?

The following table provides the popularity of typical beam and girder spans:

Table C.6: Popularity of beam and girder spans in Industrial and Office buildings

Beam span ranges (m)	Percentage in Industrial Structures	Percentage in Office Buildings
0 - 2	10 %	20 %
2 - 4	30 %	30 %
4 - 6	30 %	30 %
6 - 8	20 %	15 %
8 - 10	10 %	5 %

C.2.2.3 Engineering questions applicable to girders (primary beams)

1 How often do you use welded plate girders?

Plate girders are used approximately 5 % of the time.

2 How often do you use fixed or simply supported girders (percentages)?

The girders are almost always simply supported. Thus, 100 % of girders are simply supported.

3 Are the applied loads normally distributed or point loads on the plate girders used and what are they typically?

The applied loads are normally point loads and they can range from 10 kN to 2000 kN. The point loads normally correspond to the applied loads from the secondary beams spaced between 1.5 m and 2 m.

4 Are the girders normally laterally supported or unsupported in the structures you design (percentage)?

- Percentage laterally supported girders: 60 %
- Percentage laterally unsupported girders: 40 %

5 What are the typical spans of the girders in the structures you design and how often do they occur (percentage)?

Table C.6 presents the popularity of beam and girder spans in different structures.

6 What are the typical effective lengths (beam spacing) of the girders in the structures you design and how often do they occur (percentage)?

The effective lengths of the girders range between 1.5 and 2 m, the same as the beam spacing.

C.2.2.4 Engineering questions applicable to beams and girders

1 Is there any other information you feel I should know to obtain an optimal set of welded I-sections?

The participant did not have any additional information.

C.3 Participant 3

C.3.1 Participant information

Position at company:	Technical Director
Academic Qualifications:	MEng and PrEng (highest academic qualifications)
Experience:	37 years of experience in steel and concrete structures

C.3.2 Questions with Answers

This section contains all the information obtained from the interview with the above mentioned participant, conducted by the investigator (Mr N. Tredoux). The questions and corresponding answers were as follows:

C.3.2.1 General engineering questions

1 What are the typical steel structures you normally design and how many of them per year?

The participant normally designs 6 light industrial buildings per year. These light industrial structures (warehouses) are normally in the range of 30 000 m^2, which are considerable larger.

2 Which I-sections do you use most often and why?

For Beams (secondary beams):

Different groups of I-sections are used for short spans without floors and for composite floors, which normally has long spans. The I-sections are grouped as follows:

– Beams for short spans: 203 x 133 x 25
254 x 146 x 31
305 x 102 x 25

– Beams for long spans: 356 x 171 x 45
406 x 140 x 39
406 x 178 x 54

It is assumed that the concrete floors on metal deck provide lateral support even if they are not always used with shear studs. The metal deck of the concrete floors is welded to the beams to ensure lateral support of the floors.

For Girders (primary beams):

Steel beams are normally supported directly onto columns. Girders are thus rarely used in the structures of the participant. When girders are required, an truss would normally be used instead of hot-rolled I-sections or welded plate girders.

For Columns: 152 x 152 x 23
203 x 203 x 46
254 x 146 x 31
305 x 102 x 25

The columns of the structure are normally concrete or partly concrete with a steel column on top of a concrete column. The columns are mostly concrete, because they provide a lot of stiffness to the structure, bracing not normally used on the base of the structures. The reason being that bracing would cover truck entrances, etc.

C.3.2.2 Engineering questions applicable to beams (secondary beams)

1 How often do you use fixed or simply supported beams (percentages)?

– Percentage pinned beams: 90 - 95 %

– Percentage fixed beams: 5 - 10 %

2 What are the typical floors in the steel structures you design?

The typical floors of light industrial structures are presented in Table C.7. The concrete floors with metal deck are normally Bond-dek floors.

Table C.7: Popularity of typical light industrial floor structures according to Participant 3

Floor type	**Thickness range (mm)**	**Percentage**
Grating	40	20 %
Vastrap	4.5	20 %
Concrete on metal deck (not composite)	140	20 %
Concrete on metal deck (composite)	140	40 %

3 What are the typical imposed loads on the floors, roofs and beams (if applicable) of the structures you design?

The imposed loads include all the imposed loads in one, from the live loads specified by the loading codes to the services in the structure. The following table provides the typical imposed loads used by the participant.

Table C.8: Typical imposed loads according to Participant 3

Part of structure	**Imposed loads (kPa)**
Industrial floors	5
Industrial roofs	1

4 What are the typical wind loads on the roofs of the structures you design?

There are no typical wind loads. The wind loads are used according to the loading codes.

5 What are the typical permanent loads of the structures you design?

The permanent loads of the floors, including everything, are normally 3 kPa.

6 Are the I-beams normally laterally supported or unsupported in the structures you design (percentage)?

- Percentage laterally supported beams: 80 %
- Percentage laterally unsupported beams: 20 %

7 What are the typical spans of the I-beams in the structures you design and how often do they occur (percentage)?

The beam and girder spans are in the same ranges. Normally grids of 8.4 x 8.4 m, 8.5 x 7.8 m and 8.5 x 10 m are used for parking garages. The grids correspond to different vehicle spacings. Grids of 8.5 x 7.8 m and 8.4 x 8.4 m are also used for office buildings. The typical spans are thus 7.8, 8.4, 8.5 and 10 m.

C.3.2.3 Engineering questions applicable to girders (primary beams)

1 How often do you use welded plate girders?

Welded plate girders are rarely used.

2 How often do you use fixed or simply supported girders (percentages)?

The girders are almost always simply supported. Girders are thus 100 % simply supported.

3 Are the applied loads normally distributed or point loads on the plate girders used and what are they typically?

The girders are rarely used. No loads can thus be supplied that are normally applied to girders.

4 Are the girders normally laterally supported or unsupported in the structures you design (percentage)?

If girders are used, they are normally laterally supported.

5 What are the typical spans of the girders in the structures you design and how often do they occur (percentage)?

The girder spans are the same as used for beams (refer to Section C.3.2.2).

6 What are the typical effective lengths (beam spacing) of the girders in the structures you design and how often do they occur (percentage)?

The effective lengths of the girders range between 2.5 m and 2.8 m, the same as the beam spacing, which correspond to the span range of unpropped Bond-dek slabs.

C.3.2.4 Engineering questions applicable to beams and girders

1 Is there any other information you feel I should know to obtain an optimal set of welded I-sections?

The participant did not have any additional information.

C.4 Participant 4

C.4.1 Participant information

Position at company:	Structural Engineer
Academic Qualifications:	MEng, BEng and PrEng
Experience:	10 years of experience in steel and concrete structures

C.4.2 Questions with Answers

This section contains all the information obtained from the interview with the above mentioned participant, conducted by the investigator (Mr N. Tredoux). The questions and corresponding answers were as follows:

C.4.2.1 General engineering questions

1 What are the typical steel structures you normally design and how many of them per year?

Approximately 20 houses, 1 to 2 industrial structures and 2 to 3 office buildings or apartment buildings are designed by the participant per year.

2 Which I-sections do you use most often and why?

For Beams (secondary beams):

Different groups of I-sections are used for houses and for industrial buildings. The I-sections are grouped as follows:

- Beams used in houses: 200 IPE's
254 mm deep I-sections
- Beams used in industrial buildings: 305 mm deep I-sections
356 mm deep I-sections

Different I-sections are used for industrial structures, because the spans are normally longer.

For Girders (primary beams):

Girders are seldom used, as portal frames are normally used in industrial structures.

For Columns: 254 x 254 x 73
305 x 305 x 97
254 x 146 x 31
356 mm deep I-sections

A 254 x 254 x 73 column is normally used, because a 230 mm thick brick wall fits perfectly into the 254 mm deep hot-rolled I-sections. The 305 x 305 x 97 and 356 mm deep I-sections are normally used in industrial structures.

C.4.2.2 Engineering questions applicable to beams (secondary beams)

1 How often do you use fixed or simply supported beams (percentages)?

- Percentage pinned beams: 80 %
- Percentage fixed beams: 20 %

2 What are the typical floors in the steel structures you design?

The industrial structures normally designed, are used for offices or storage space. The same floor systems are normally used for these industrial structures, as for houses. All the floors are hollow core or in-situ concrete floors. The hollow core floors normally range between 170 to 255 mm and the in-situ floors the same. Concrete floors with a metal deck are not used much.

3 What are the typical imposed loads on the floors, roofs and beams (if applicable) of the structures you design?

The typical imposed loads the participant uses are presented in the table below:

Table C.9: Typical imposed loads according to Participant 4

Part of structure	**Imposed loads (kPa)**	**Other imposed loads (kPa)**
Industrial storage space	5	1 -1.5 (Services)
Flat roofs	1.5 - 3	

4 What are the typical wind loads on the roofs of the structures you design?

The typical wind loads ranges between 1 kPa and 2 kPa.

5 What are the typical permanent loads of the structures you design?

The permanent loads account for the permanent loads of the floors and normally include brick walls as an extra permanent load. The brick walls normally add permanent loads of 18 kN/m.

6 Are the I-beams normally laterally supported or unsupported in the structures you design (percentage)?

- Percentage laterally supported beams: 20 %
- Percentage laterally unsupported beams: 80 %

7 What are the typical spans of the I-beams in the structures you design and how often do they occur (percentage)?

The I-beams normally span between 6 m and 10 m for houses and for industrial structures between 5 m and 15 m. The 5 m spans are normally used in small industrial buildings and 15 m spans in large industrial buildings.

C.4.2.3 Engineering questions applicable to girders (primary beams)

Girders are not used.

C.4.2.4 Engineering questions applicable to beams and girders

1 Is there any other information you feel I should know to obtain an optimal set of welded I-sections?

The participant did not have any additional information.

Appendix D

Calculation of weighting factors

D.1 Weighting B

D.1.1 Weighting factors for lateral support conditions

Table D.1: Calculations to determine the popularity of lateral support conditions of Weighting B

		Laterally supported		**Laterally unsupported**	
Participants	**Number of Structures**	**Frequency (%)**	**Weighted frequency**	**Frequency (%)**	**Weighted frequency**
Participant 1	50	5	2.5	95	47.5
Participant 2	40	70	28	30	12
Participant 3	6	80	4.8	20	1.2
Participant 4	5	20	1	80	4
		Total	36.3	**Total**	64.7
		Total %	35.94	**Total %**	64.06
		$f_{lateral}$	0.36	$f_{lateral}$	0.64

D.1.2 Weighting factors for span

Refer to Table D.2 for the calculations on span weighting factors for Weighting B.

Table D.2: Calculations to determine the span weighting factors of Weighting B

	Frequency of beam spans (%)				Popularity of beam spans out of 1.0					
Structures	Industrial		Office blocks	Light industrial	Industrial		Office blocks	Light industrial	Combined	Changed
Beam span (m)	Participant 1	Participant 2	Participant 2	Participant 3	Participant 1	Participant 2	Participant 2	Participant 3		
0	0	0	0	0	0.00	0.00	0.00	0.00	0.00	0.00
0.5	0	10	20	0	0.00	0.33	0.67	0.00	0.17	0.09
1	0	10	20	0	0.00	0.33	0.67	0.00	0.17	0.17
1.5	0	10	20	0	0.00	0.33	0.67	0.00	0.17	0.56
2	80	30	30	0	1.00	1.00	1.00	0.00	0.94	0.94
2.5	80	30	30	0	1.00	1.00	1.00	0.00	0.94	0.94
3	80	30	30	0	1.00	1.00	1.00	0.00	0.94	0.94
3.5	20	30	30	0	0.25	1.00	1.00	0.00	0.55	0.55
4	20	30	30	0	0.25	1.00	1.00	0.00	0.55	0.48
4.5	0	30	30	0	0.00	1.00	1.00	0.00	0.42	0.42
5	0	30	30	0	0.00	1.00	1.00	0.00	0.42	0.39
5.5	0	30	30	0	0.00	1.00	1.00	0.00	0.42	0.37
6	0	20	15	0	0.00	0.67	0.50	0.00	0.26	0.34
6.5	0	20	15	0	0.00	0.67	0.50	0.00	0.26	0.32
7	0	20	15	0	0.00	0.67	0.50	0.00	0.26	0.29
7.5	0	20	15	70	0.00	0.67	0.50	1.00	0.32	0.27
8	0	10	5	70	0.00	0.33	0.17	1.00	0.18	0.25
8.5	0	10	5	70	0.00	0.33	0.17	1.00	0.18	0.22
9	0	10	5	30	0.00	0.33	0.17	0.43	0.15	0.20
9.5	0	10	5	30	0.00	0.33	0.17	0.43	0.15	0.17
10	0	10	5	30	0.00	0.33	0.17	0.43	0.15	0.15

D.1.3 Weighting factors for loading

Table D.3: Number of structures considered in the load weighting factor calculations of Weighting B

	No. of structures per participant					
Structures	**1**	**2**	**3**	**4**	**Total no. of structures**	**% of all structures**
Industrial	50	30			80	80.81
Light industrial			6		6	6.06
Office buildings		10		3	13	13.13

D.2 Weighting C

D.2.1 Weighting factors for lateral support conditions

Table D.4: Calculations to determine the popularity of lateral support conditions of Weighting C

		Laterally supported		Laterally unsupported	
Participants	**Number of Structures**	**Frequency (%)**	**Weighted frequency**	**Frequency (%)**	**Weighted frequency**
Participant 1	50	0	0	100	50
Participant 2	40	60	24	40	16
		Total	24	**Total**	66
		Total %	26.67	**Total %**	73.33
		$f_{lateral}$	0.27	$f_{lateral}$	0.73

D.2.2 Weighting factors for span

Refer to Table D.5 for the calculations on span weighting factors for Weighting C.

Table D.5: Calculations to determine the span weighting factors of Weighting C

Structures	**Frequency of girder spans (%)**			**Popularity of girder spans out of 1.0**				
	Industrial		Office blocks	Industrial		Office blocks	Combined	Changed
Beam span (m)	Participant 1	Participant 2	Participant 2	Participant 1	Participant 2	Participant 2		
0	0	0	0	0.00	0.00	0.00	0.00	0.00
0.5	0	0	0	0.00	0.00	0.00	0.00	0.00
1	0	10	20	0.00	0.33	0.67	0.19	0.13
1.5	0	10	20	0.00	0.33	0.67	0.19	0.25
2	0	30	30	0.00	1.00	1.00	0.44	0.38
2.5	0	30	30	0.00	1.00	1.00	0.44	0.50
3	20	30	30	0.33	1.00	1.00	0.63	0.63
3.5	20	30	30	0.33	1.00	1.00	0.63	0.63
4	20	30	30	0.33	1.00	1.00	0.63	0.63
4.5	60	30	30	1.00	1.00	1.00	1.00	1.00
5	60	30	30	1.00	1.00	1.00	1.00	1.00
5.5	60	30	30	1.00	1.00	1.00	1.00	1.00
6	20	20	15	0.33	0.67	0.50	0.46	0.46
6.5	20	20	15	0.33	0.67	0.50	0.46	0.46
7	20	20	15	0.33	0.67	0.50	0.46	0.46
7.5	0	20	15	0.00	0.67	0.50	0.28	0.28
8	0	10	5	0.00	0.33	0.17	0.13	0.13
8.5	0	10	5	0.00	0.33	0.17	0.13	0.13
9	0	10	5	0.00	0.33	0.17	0.13	0.13
9.5	0	10	5	0.00	0.33	0.17	0.13	0.13
10	0	10	5	0.00	0.33	0.17	0.13	0.13

D.2.3 Weighting factors for loading

Table D.6: Number of structures considered in load weighting factor calculations of Weighting C

	No. of structures per participant				
Structures	**1**	**2**	**4**	**Total no. of structures**	**% of all structures**
Industrial	50	30		80	86.02
Office buildings		10	3	13	13.98

Appendix E

Optimal sets of I-sections

E.1 Optimal sets for beam conditions

E.1.1 Based on Initial Set 1

Table E.1: Sections present in all the final A1 and B1 Optimal Sets with a minimum rating between 75 and 95

Section Name $h_w^* \times b \times m$	h_w^* **mm**	b **mm**	t_w **mm**	t_f **mm**
160.0 x 105.0 x 19.47	160	105	5	8
175.0 x 150.0 x 31.79	175	150	6	10
200.0 x 110.0 x 21.67	200	110	5	8
240.0 x 100.0 x 17.27	240	100	5	5
240.0 x 150.0 x 34.85	240	150	6	10
240.0 x 150.0 x 39.56	240	150	6	12
250.0 x 135.0 x 26.77	250	135	5	8
270.0 x 200.0 x 54.64	270	200	8	12
280.0 x 150.0 x 29.83	280	150	5	8
280.0 x 150.0 x 36.74	280	150	6	10
280.0 x 255.0 x 73.63	280	255	8	14
290.0 x 100.0 x 20.8	290	100	5	6
290.0 x 100.0 x 23.08	290	100	6	6
290.0 x 105.0 x 26.85	290	105	6	8
290.0 x 165.0 x 44.74	290	165	6	12
325.0 x 175.0 x 42.78	325	175	6	10
330.0 x 130.0 x 31.87	330	130	6	8

Note : Sections only in A1 Optimal Sets = blue
Sections only in B1 Optimal Sets = green

Table E.1 (continued)

Section Name $h_w^* \times b \times m$	h_w^* **mm**	b **mm**	t_w **mm**	t_f **mm**
335.0 x 170.0 x 47.81	335	170	6	12
360.0 x 300.0 x 103.62	360	300	10	16
370.0 x 200.0 x 60.92	370	200	8	12
370.0 x 200.0 x 67.2	370	200	8	14
380.0 x 140.0 x 35.48	380	140	6	8
380.0 x 140.0 x 44.27	380	140	6	12
380.0 x 180.0 x 52.12	380	180	8	10
400.0 x 300.0 x 141.3	400	300	12	22
430.0 x 150.0 x 50.55	430	150	8	10
450.0 x 300.0 x 127.17	450	300	12	18
470.0 x 200.0 x 67.2	470	200	8	12
470.0 x 200.0 x 73.48	470	200	8	14
500.0 x 165.0 x 62.49	500	165	8	12
545.0 x 300.0 x 136.12	545	300	12	18
575.0 x 180.0 x 79.05	575	180	10	12

Note : Sections only in A1 Optimal Sets = blue
Sections only in B1 Optimal Sets = green

Table E.2: Most optimal set of the final A1 Optimal Sets

Section Name $h_w^* \times b \times m$	**Final ranking**	h_w^* **mm**	b **mm**	t_w **mm**	t_f **mm**
140.0 x 100.0 x 22.29	96.00	140	100	6	10
160.0 x 105.0 x 19.47	2584.67	160	105	5	8
175.0 x 150.0 x 31.79	230.00	175	150	6	10
190.0 x 100.0 x 18.37	770.78	190	100	6	6
190.0 x 130.0 x 29.36	2292.77	190	130	6	10
190.0 x 135.0 x 25.91	1646.13	190	135	6	8
200.0 x 110.0 x 21.67	1660.00	200	110	5	8
225.0 x 200.0 x 51.81	1604.76	225	200	8	12
230.0 x 125.0 x 24.73	3790.56	230	125	5	8
240.0 x 100.0 x 17.27	18121.38	240	100	5	5
240.0 x 100.0 x 20.72	2776.54	240	100	6	6
240.0 x 150.0 x 34.85	3881.19	240	150	6	10

Table E.2 (continued)

Section Name $h_w^* \times b \times m$	**Final ranking**	h_w^* **mm**	b **mm**	t_w **mm**	t_f **mm**
240.0 x 150.0 x 39.56	855.33	240	150	6	12
250.0 x 135.0 x 26.77	2924.11	250	135	5	8
270.0 x 200.0 x 54.64	4611.94	270	200	8	12
280.0 x 150.0 x 29.83	2754.96	280	150	5	8
280.0 x 150.0 x 36.74	887.08	280	150	6	10
280.0 x 200.0 x 61.54	2502.51	280	200	8	14
280.0 x 255.0 x 73.63	2365.58	280	255	8	14
280.0 x 255.0 x 86.04	2389.16	280	255	10	16
290.0 x 100.0 x 20.8	6522.15	290	100	5	6
290.0 x 100.0 x 23.08	948.55	290	100	6	6
290.0 x 100.0 x 29.36	2871.70	290	100	6	10
290.0 x 105.0 x 26.85	3169.93	290	105	6	8
290.0 x 105.0 x 30.14	268.75	290	105	6	10
290.0 x 165.0 x 39.56	620.32	290	165	6	10
290.0 x 165.0 x 44.74	1552.22	290	165	6	12
305.0 x 160.0 x 39.49	3425.30	305	160	6	10
320.0 x 200.0 x 64.06	674.83	320	200	8	14
320.0 x 200.0 x 70.34	3212.54	320	200	8	16
320.0 x 250.0 x 87.92	3736.75	320	250	10	16
320.0 x 255.0 x 97.18	172.50	320	255	10	18
325.0 x 175.0 x 42.78	1287.76	325	175	6	10
325.0 x 175.0 x 53.38	411.54	325	175	8	12
330.0 x 125.0 x 31.24	4984.59	330	125	6	8
330.0 x 125.0 x 35.17	3436.65	330	125	6	10
330.0 x 130.0 x 31.87	705.79	330	130	6	8
330.0 x 170.0 x 42.23	2883.83	330	170	6	10
335.0 x 170.0 x 47.81	2352.43	335	170	6	12
360.0 x 300.0 x 103.62	1742.69	360	300	10	16
370.0 x 200.0 x 60.92	2742.11	370	200	8	12
370.0 x 200.0 x 67.2	1172.47	370	200	8	14
380.0 x 140.0 x 35.48	4750.82	380	140	6	8
380.0 x 140.0 x 44.27	6316.37	380	140	6	12

Table E.2 (continued)

Section Name $h_w^* \times b \times m$	**Final ranking**	h_w^* **mm**	b **mm**	t_w **mm**	t_f **mm**
380.0 x 140.0 x 50.24	1025.74	380	140	8	12
380.0 x 180.0 x 57.78	411.22	380	180	8	12
380.0 x 260.0 x 95.14	1544.24	380	260	10	16
400.0 x 300.0 x 141.3	6.39	400	300	12	22
430.0 x 150.0 x 50.55	3325.35	430	150	8	10
430.0 x 150.0 x 59.97	3701.56	430	150	8	14
430.0 x 280.0 x 112.88	253.97	430	280	10	18
450.0 x 300.0 x 117.75	79.11	450	300	12	16
450.0 x 300.0 x 127.17	157.38	450	300	12	18
470.0 x 200.0 x 67.2	72.67	470	200	8	12
470.0 x 200.0 x 73.48	168.75	470	200	8	14
500.0 x 165.0 x 62.49	2379.56	500	165	8	12
500.0 x 210.0 x 72.22	324.28	500	210	10	10
500.0 x 210.0 x 85.41	327.56	500	210	10	14
545.0 x 300.0 x 136.12	17.36	545	300	12	18
570.0 x 180.0 x 89.96	6.39	570	180	10	16
575.0 x 180.0 x 79.05	308.49	575	180	10	12

Table E.3: Most optimal set of the final B1 Optimal Sets

Section Name $h_w^* \times b \times m$	**Final ranking**	h_w^* **mm**	b **mm**	t_w **mm**	t_f **mm**
80.0 x 100.0 x 15.7	7082.65	80	100	5	8
140.0 x 100.0 x 19.15	1180.60	140	100	6	8
140.0 x 100.0 x 22.29	21.28	140	100	6	10
160.0 x 105.0 x 19.47	324.59	160	105	5	8
175.0 x 150.0 x 31.79	40.65	175	150	6	10
190.0 x 135.0 x 25.91	313.84	190	135	6	8
200.0 x 110.0 x 21.67	471.44	200	110	5	8
230.0 x 125.0 x 24.73	897.19	230	125	5	8
235.0 x 150.0 x 34.62	559.94	235	150	6	10

Table E.3 (continued)

Section Name $h_w^* \times b \times m$	**Final ranking**	h_w^* **mm**	b **mm**	t_w **mm**	t_f **mm**
240.0 x 100.0 x 17.27	5878.03	240	100	5	5
240.0 x 150.0 x 34.85	32.27	240	150	6	10
240.0 x 150.0 x 39.56	58.44	240	150	6	12
250.0 x 135.0 x 26.77	361.68	250	135	5	8
250.0 x 135.0 x 32.97	57.61	250	135	6	10
270.0 x 200.0 x 54.64	438.62	270	200	8	12
280.0 x 150.0 x 29.83	1041.09	280	150	5	8
280.0 x 150.0 x 36.74	122.17	280	150	6	10
280.0 x 200.0 x 61.54	337.62	280	200	8	14
280.0 x 255.0 x 73.63	312.65	280	255	8	14
285.0 x 165.0 x 39.33	593.89	285	165	6	10
285.0 x 165.0 x 54.16	422.83	285	165	8	14
290.0 x 100.0 x 20.8	879.29	290	100	5	6
290.0 x 100.0 x 23.08	85.10	290	100	6	6
290.0 x 105.0 x 26.85	256.65	290	105	6	8
290.0 x 105.0 x 30.14	19.29	290	105	6	10
290.0 x 165.0 x 44.74	219.17	290	165	6	12
320.0 x 200.0 x 64.06	73.44	320	200	8	14
320.0 x 200.0 x 70.34	383.29	320	200	8	16
320.0 x 250.0 x 87.92	682.33	320	250	10	16
320.0 x 255.0 x 89.18	50.79	320	255	10	16
320.0 x 255.0 x 97.18	38.39	320	255	10	18
325.0 x 175.0 x 42.78	169.82	325	175	6	10
330.0 x 125.0 x 31.24	381.95	330	125	6	8
330.0 x 130.0 x 31.87	51.94	330	130	6	8
330.0 x 170.0 x 42.23	345.73	330	170	6	10
335.0 x 170.0 x 47.81	326.60	335	170	6	12
360.0 x 300.0 x 103.62	218.57	360	300	10	16
370.0 x 200.0 x 60.92	313.50	370	200	8	12
370.0 x 200.0 x 67.2	136.55	370	200	8	14
380.0 x 140.0 x 35.48	573.88	380	140	6	8
380.0 x 140.0 x 44.27	423.75	380	140	6	12

Table E.3 (continued)

Section Name $h_w^* \times b \times m$	**Final ranking**	h_w^* **mm**	b **mm**	t_w **mm**	t_f **mm**
380.0 x 180.0 x 52.12	0.80	380	180	8	10
380.0 x 260.0 x 95.14	123.70	380	260	10	16
380.0 x 265.0 x 133.29	1.28	380	265	14	22
400.0 x 300.0 x 141.3	6.14	400	300	12	22
430.0 x 150.0 x 50.55	273.78	430	150	8	10
430.0 x 150.0 x 59.97	249.98	430	150	8	14
450.0 x 300.0 x 117.75	53.61	450	300	12	16
450.0 x 300.0 x 127.17	24.31	450	300	12	18
470.0 x 200.0 x 67.2	3.86	470	200	8	12
470.0 x 200.0 x 73.48	39.70	470	200	8	14
490.0 x 300.0 x 121.52	10.62	490	300	12	16
500.0 x 165.0 x 62.49	158.80	500	165	8	12
500.0 x 165.0 x 75.52	7.48	500	165	10	14
545.0 x 300.0 x 136.12	6.24	545	300	12	18
570.0 x 180.0 x 89.96	3.59	570	180	10	16
575.0 x 180.0 x 79.05	28.63	575	180	10	12

E.1.2 Based on Initial Set 2

Table E.4: Sections present in three or more of the final A2 and B2 Optimal Sets with a minimum rating between 75 and 95

Section Name $h_w^* \times b \times m$	h_w^* **mm**	b **mm**	t_w **mm**	t_f **mm**
160.0 x 120.0 x 21.35	160	120	5	8
170.0 x 100.0 x 14.52	170	100	5	5
190.0 x 120.0 x 18.76	190	120	5	6
190.0 x 160.0 x 27.55	190	160	5	8
200.0 x 100.0 x 15.7	200	100	5	5
210.0 x 160.0 x 33.36	210	160	5	10
220.0 x 100.0 x 16.48	220	100	5	5
220.0 x 170.0 x 35.33	220	170	5	10

Note : Sections only in A2 Optimal Sets = blue
Sections only in B2 Optimal Sets = green

Table E.4 (continued)

Section Name $h_w^* \times b \times m$	h_w^* **mm**	b **mm**	t_w **mm**	t_f **mm**
230.0 x 190.0 x 38.86	230	190	5	10
240.0 x 100.0 x 17.27	240	100	5	5
260.0 x 160.0 x 30.3	260	160	5	8
260.0 x 210.0 x 43.17	260	210	5	10
270.0 x 100.0 x 18.45	270	100	5	5
270.0 x 220.0 x 52.05	270	220	5	12
300.0 x 250.0 x 58.88	300	250	5	12
310.0 x 100.0 x 20.02	310	100	5	5
320.0 x 260.0 x 69.71	320	260	5	14
340.0 x 100.0 x 21.2	340	100	5	5
340.0 x 280.0 x 74.89	340	280	5	14
350.0 x 100.0 x 21.59	350	100	5	5
370.0 x 210.0 x 47.49	370	210	5	10
380.0 x 100.0 x 22.77	380	100	5	5
400.0 x 100.0 x 23.55	400	100	5	5
570.0 x 330.0 x 105.27	570	330	5	16
580.0 x 100.0 x 30.61	580	100	5	5
660.0 x 110.0 x 36.27	660	110	5	6
720.0 x 330.0 x 111.16	720	330	5	16

Note : Sections only in A2 Optimal Sets = blue
Sections only in B2 Optimal Sets = green

Table E.5: Most optimal set of the final A2 Optimal Sets

Section Name $h_w^* \times b \times m$	**Final ranking**	h_w^* **mm**	b **mm**	t_w **mm**	t_f **mm**
150.0 x 100.0 x 15.31	704.01	150	100	5	6
150.0 x 110.0 x 19.7	986.63	150	110	5	8
160.0 x 100.0 x 15.7	170.56	160	100	5	6
160.0 x 110.0 x 16.64	1030.79	160	110	5	6
160.0 x 120.0 x 17.58	1268.28	160	120	5	6
160.0 x 120.0 x 21.35	2097.63	160	120	5	8
170.0 x 100.0 x 16.09	89.27	170	100	5	6
170.0 x 130.0 x 23	2139.96	170	130	5	8

Table E.5 (continued)

Section Name $h_w^* \times b \times m$	**Final ranking**	h_w^* **mm**	b **mm**	t_w **mm**	t_f **mm**
180.0 x 100.0 x 14.91	9647.69	180	100	5	5
180.0 x 140.0 x 24.65	2491.28	180	140	5	8
190.0 x 120.0 x 18.76	786.83	190	120	5	6
190.0 x 150.0 x 26.3	2372.25	190	150	5	8
200.0 x 100.0 x 15.7	2383.67	200	100	5	5
200.0 x 150.0 x 31.4	1022.00	200	150	5	10
200.0 x 160.0 x 27.95	2514.85	200	160	5	8
220.0 x 100.0 x 16.48	2425.49	220	100	5	5
220.0 x 160.0 x 33.76	2895.08	220	160	5	10
220.0 x 170.0 x 35.33	2574.01	220	170	5	10
230.0 x 180.0 x 37.29	2641.81	230	180	5	10
230.0 x 190.0 x 38.86	2178.11	230	190	5	10
240.0 x 100.0 x 17.27	2757.43	240	100	5	5
240.0 x 200.0 x 40.82	2631.47	240	200	5	10
260.0 x 160.0 x 30.3	2154.92	260	160	5	8
260.0 x 210.0 x 43.17	3156.31	260	210	5	10
260.0 x 210.0 x 49.77	1973.22	260	210	5	12
270.0 x 100.0 x 18.45	4110.08	270	100	5	5
270.0 x 220.0 x 52.05	2601.30	270	220	5	12
290.0 x 230.0 x 54.71	2674.95	290	230	5	12
290.0 x 240.0 x 56.6	1961.22	290	240	5	12
310.0 x 100.0 x 20.02	5480.59	310	100	5	5
310.0 x 210.0 x 45.14	1536.31	310	210	5	10
310.0 x 250.0 x 59.27	2410.79	310	250	5	12
340.0 x 280.0 x 74.89	2877.26	340	280	5	14
350.0 x 100.0 x 21.59	5267.45	350	100	5	5
350.0 x 290.0 x 77.48	1060.16	350	290	5	14
370.0 x 210.0 x 47.49	1976.49	370	210	5	10
380.0 x 100.0 x 22.77	3907.05	380	100	5	5
390.0 x 250.0 x 62.41	2261.62	390	250	5	12
400.0 x 100.0 x 23.55	2406.93	400	100	5	5
420.0 x 100.0 x 24.34	2476.62	420	100	5	5

Table E.5 (continued)

Section Name $h_w^* \times b \times m$	**Final ranking**	h_w^* **mm**	b **mm**	t_w **mm**	t_f **mm**
420.0 x 330.0 x 99.38	157.55	420	330	5	16
440.0 x 290.0 x 81.01	911.35	440	290	5	14
460.0 x 100.0 x 25.91	4494.01	460	100	5	5
510.0 x 100.0 x 27.87	5175.31	510	100	5	5
510.0 x 250.0 x 67.12	2542.92	510	250	5	12
530.0 x 290.0 x 84.54	648.94	530	290	5	14
570.0 x 250.0 x 69.47	1124.65	570	250	5	12
580.0 x 100.0 x 30.61	5650.60	580	100	5	5
590.0 x 110.0 x 33.52	4489.81	590	110	5	6
610.0 x 330.0 x 106.84	33.10	610	330	5	16
640.0 x 100.0 x 34.54	1244.16	640	100	5	6
640.0 x 290.0 x 88.86	521.35	640	290	5	14
660.0 x 110.0 x 36.27	1970.71	660	110	5	6
670.0 x 100.0 x 38.86	2113.34	670	100	5	8
690.0 x 110.0 x 40.9	1194.33	690	110	5	8
750.0 x 130.0 x 45.77	1332.77	750	130	5	8
750.0 x 150.0 x 48.28	213.59	750	150	5	8
770.0 x 290.0 x 93.96	311.03	770	290	5	14
790.0 x 110.0 x 48.28	213.59	790	110	5	10
830.0 x 110.0 x 53.3	68.11	830	110	5	12

Table E.6: Most optimal set of the final B2 Optimal Sets

Section Name $h_w^* \times b \times m$	**Final ranking**	h_w^* **mm**	b **mm**	t_w **mm**	t_f **mm**
150.0 x 110.0 x 19.7	220.53	150	110	5	8
150.0 x 120.0 x 17.19	453.98	150	120	5	6
160.0 x 110.0 x 16.64	1061.26	160	110	5	6
160.0 x 120.0 x 21.35	466.49	160	120	5	8
170.0 x 100.0 x 14.52	11063.76	170	100	5	5
170.0 x 120.0 x 17.98	217.37	170	120	5	6
170.0 x 130.0 x 23	478.67	170	130	5	8
180.0 x 140.0 x 24.65	475.76	180	140	5	8

Table E.6 (continued)

Section Name $h_w^* \times b \times m$	**Final ranking**	h_w^* **mm**	b **mm**	t_w **mm**	t_f **mm**
190.0 x 100.0 x 15.31	608.10	190	100	5	5
190.0 x 150.0 x 26.3	442.00	190	150	5	8
200.0 x 120.0 x 19.15	245.47	200	120	5	6
200.0 x 150.0 x 31.4	179.16	200	150	5	10
200.0 x 160.0 x 27.95	438.41	200	160	5	8
220.0 x 160.0 x 33.76	396.47	220	160	5	10
220.0 x 170.0 x 35.33	364.55	220	170	5	10
230.0 x 180.0 x 37.29	393.99	230	180	5	10
230.0 x 190.0 x 38.86	317.16	230	190	5	10
240.0 x 100.0 x 17.27	830.39	240	100	5	5
240.0 x 120.0 x 20.72	100.38	240	120	5	6
250.0 x 200.0 x 41.21	411.26	250	200	5	10
260.0 x 160.0 x 30.3	314.22	260	160	5	8
260.0 x 210.0 x 43.17	345.93	260	210	5	10
260.0 x 210.0 x 49.77	274.37	260	210	5	12
270.0 x 100.0 x 18.45	410.88	270	100	5	5
270.0 x 220.0 x 52.05	303.68	270	220	5	12
280.0 x 120.0 x 22.29	2.84	280	120	5	6
290.0 x 120.0 x 22.69	4.80	290	120	5	6
290.0 x 240.0 x 56.6	550.77	290	240	5	12
300.0 x 250.0 x 58.88	255.47	300	250	5	12
310.0 x 100.0 x 20.02	499.50	310	100	5	5
310.0 x 160.0 x 32.26	62.00	310	160	5	8
320.0 x 260.0 x 69.71	176.73	320	260	5	14
340.0 x 270.0 x 72.69	180.93	340	270	5	14
340.0 x 280.0 x 74.89	124.44	340	280	5	14
350.0 x 100.0 x 21.59	451.84	350	100	5	5
360.0 x 250.0 x 61.23	176.78	360	250	5	12
370.0 x 210.0 x 47.49	440.78	370	210	5	10
380.0 x 100.0 x 22.77	310.99	380	100	5	5
400.0 x 100.0 x 23.55	193.75	400	100	5	5
410.0 x 330.0 x 98.99	24.17	410	330	5	16

Table E.6 (continued)

Section Name $h_w^* \times b \times m$	**Final ranking**	h_w^* **mm**	b **mm**	t_w **mm**	t_f **mm**
420.0 x 100.0 x 24.34	188.28	420	100	5	5
420.0 x 290.0 x 80.23	213.14	420	290	5	14
450.0 x 250.0 x 64.76	232.18	450	250	5	12
460.0 x 100.0 x 25.91	334.40	460	100	5	5
490.0 x 100.0 x 27.08	228.03	490	100	5	5
490.0 x 290.0 x 82.97	70.35	490	290	5	14
510.0 x 100.0 x 27.87	146.70	510	100	5	5
510.0 x 250.0 x 67.12	137.56	510	250	5	12
510.0 x 330.0 x 102.91	13.79	510	330	5	16
530.0 x 100.0 x 28.65	128.53	530	100	5	5
550.0 x 100.0 x 29.44	117.53	550	100	5	5
570.0 x 100.0 x 30.22	96.08	570	100	5	5
610.0 x 190.0 x 53.77	0.16	610	190	5	10
620.0 x 190.0 x 54.16	0.39	620	190	5	10
630.0 x 190.0 x 54.56	0.62	630	190	5	10
640.0 x 100.0 x 32.97	288.44	640	100	5	5
640.0 x 100.0 x 34.54	133.00	640	100	5	6
640.0 x 190.0 x 54.95	1.70	640	190	5	10
640.0 x 290.0 x 88.86	108.16	640	290	5	14
650.0 x 190.0 x 55.34	1.73	650	190	5	10
660.0 x 110.0 x 36.27	109.19	660	110	5	6
670.0 x 190.0 x 56.13	1.82	670	190	5	10
680.0 x 120.0 x 37.99	77.90	680	120	5	6
680.0 x 120.0 x 41.76	53.83	680	120	5	8
690.0 x 100.0 x 39.64	62.55	690	100	5	8
690.0 x 200.0 x 58.48	1.30	690	200	5	10
710.0 x 100.0 x 43.57	32.42	710	100	5	10
720.0 x 210.0 x 61.23	0.07	720	210	5	10
720.0 x 330.0 x 111.16	7.01	720	330	5	16
750.0 x 130.0 x 45.77	24.97	750	130	5	8
800.0 x 290.0 x 95.14	67.16	800	290	5	14
820.0 x 140.0 x 49.77	22.64	820	140	5	8

E.1.3 Based on Initial Set 3

Table E.7: Sections present in four or more of the final A3 and B3 Optimal Sets with a minimum rating between 75 and 95

Section Name $h_w^* \times b \times m$	h_w^* **mm**	b **mm**	t_w **mm**	t_f **mm**
150.0 x 120.0 x 17.19	150	120	5	6
200.0 x 100.0 x 15.7	200	100	5	5
200.0 x 120.0 x 19.15	200	120	5	6
200.0 x 120.0 x 22.92	200	120	5	8
200.0 x 140.0 x 25.43	200	140	5	8
200.0 x 160.0 x 27.95	200	160	5	8
200.0 x 160.0 x 32.97	200	160	5	10
250.0 x 100.0 x 17.66	250	100	5	5
250.0 x 120.0 x 21.12	250	120	5	6
250.0 x 160.0 x 29.91	250	160	5	8
250.0 x 160.0 x 34.93	250	160	5	10
250.0 x 180.0 x 38.07	250	180	5	10
250.0 x 200.0 x 41.21	250	200	5	10
300.0 x 100.0 x 19.62	300	100	5	5
300.0 x 120.0 x 23.08	300	120	5	6
300.0 x 160.0 x 31.87	300	160	5	8
300.0 x 200.0 x 43.17	300	200	5	10
300.0 x 220.0 x 53.22	300	220	5	12
300.0 x 240.0 x 56.99	300	240	5	12
350.0 x 100.0 x 21.59	350	100	5	5
350.0 x 120.0 x 25.04	350	120	5	6
350.0 x 200.0 x 51.42	350	200	5	12
350.0 x 260.0 x 70.89	350	260	5	14
350.0 x 280.0 x 75.28	350	280	5	14
400.0 x 100.0 x 23.55	400	100	5	5
400.0 x 240.0 x 60.92	400	240	5	12
450.0 x 100.0 x 25.51	450	100	5	5
500.0 x 100.0 x 27.48	500	100	5	5
550.0 x 100.0 x 29.44	550	100	5	5

Note : Sections only in A3 Optimal Sets = blue
Sections only in B3 Optimal Sets = green

Table E.7 (continued)

Section Name $h_w^* \times b \times m$	h_w^* **mm**	b **mm**	t_w **mm**	t_f **mm**
550.0 x 240.0 x 66.8	550	240	5	12
600.0 x 100.0 x 31.4	600	100	5	5
600.0 x 100.0 x 32.97	600	100	5	6
650.0 x 100.0 x 38.07	650	100	5	8
650.0 x 120.0 x 36.82	650	120	5	6
650.0 x 280.0 x 87.06	650	280	5	14
700.0 x 100.0 x 40.03	700	100	5	8
700.0 x 320.0 x 107.86	700	320	5	16

Note : Sections only in A3 Optimal Sets = blue
Sections only in B3 Optimal Sets = green

Table E.8: Most optimal set of the final A3 Optimal Sets

Section Name $h_w^* \times b \times m$	**Final ranking**	h_w^* **mm**	b **mm**	t_w **mm**	t_f **mm**
150.0 x 100.0 x 13.74	6282.89	150	100	5	5
150.0 x 100.0 x 15.31	2191.57	150	100	5	6
150.0 x 100.0 x 16.48	353.22	150	100	6	6
150.0 x 100.0 x 18.45	261.15	150	100	5	8
150.0 x 100.0 x 21.59	2367.48	150	100	5	10
150.0 x 120.0 x 17.19	1978.02	150	120	5	6
150.0 x 120.0 x 22.14	98.00	150	120	6	8
150.0 x 140.0 x 23.47	146.00	150	140	5	8
200.0 x 100.0 x 15.7	4260.91	200	100	5	5
200.0 x 100.0 x 17.27	1647.14	200	100	5	6
200.0 x 120.0 x 19.15	975.42	200	120	5	6
200.0 x 120.0 x 22.92	1231.96	200	120	5	8
200.0 x 120.0 x 24.49	246.10	200	120	6	8
200.0 x 140.0 x 25.43	4005.26	200	140	5	8
200.0 x 140.0 x 29.83	713.99	200	140	5	10
200.0 x 160.0 x 27.95	4285.72	200	160	5	8
200.0 x 160.0 x 32.97	3515.42	200	160	5	10
200.0 x 180.0 x 36.11	701.63	200	180	5	10

Table E.8 (continued)

Section Name $h_w^* \times b \times m$	**Final ranking**	h_w^* **mm**	b **mm**	t_w **mm**	t_f **mm**
250.0 x 100.0 x 17.66	4880.23	250	100	5	5
250.0 x 160.0 x 29.91	1046.67	250	160	5	8
250.0 x 160.0 x 34.93	1685.91	250	160	5	10
250.0 x 180.0 x 38.07	3402.14	250	180	5	10
250.0 x 200.0 x 41.21	4756.20	250	200	5	10
300.0 x 100.0 x 19.62	6803.98	300	100	5	5
300.0 x 160.0 x 36.9	798.40	300	160	5	10
300.0 x 200.0 x 49.45	2201.98	300	200	5	12
300.0 x 220.0 x 53.22	3370.04	300	220	5	12
300.0 x 240.0 x 56.99	4096.33	300	240	5	12
350.0 x 100.0 x 21.59	6709.69	350	100	5	5
350.0 x 200.0 x 45.14	3137.46	350	200	5	10
350.0 x 200.0 x 51.42	1360.89	350	200	5	12
350.0 x 240.0 x 58.95	1304.25	350	240	5	12
350.0 x 260.0 x 70.89	1797.08	350	260	5	14
350.0 x 280.0 x 75.28	1463.23	350	280	5	14
400.0 x 100.0 x 23.55	6313.97	400	100	5	5
400.0 x 100.0 x 25.12	3118.93	400	100	5	6
400.0 x 200.0 x 47.1	1529.12	400	200	5	10
400.0 x 240.0 x 60.92	1298.33	400	240	5	12
400.0 x 260.0 x 72.85	826.23	400	260	5	14
400.0 x 280.0 x 77.24	708.68	400	280	5	14
400.0 x 320.0 x 96.08	92.19	400	320	5	16
450.0 x 100.0 x 25.51	2746.95	450	100	5	5
450.0 x 100.0 x 27.08	3397.80	450	100	5	6
450.0 x 240.0 x 62.88	1154.59	450	240	5	12
450.0 x 280.0 x 79.21	510.72	450	280	5	14
450.0 x 300.0 x 93.02	131.35	450	300	5	16
450.0 x 320.0 x 98.05	58.50	450	320	5	16
500.0 x 100.0 x 27.48	2043.13	500	100	5	5
500.0 x 240.0 x 64.84	1094.42	500	240	5	12
500.0 x 300.0 x 94.98	109.06	500	300	5	16

Table E.8 (continued)

Section Name $h_w^* \times b \times m$	**Final ranking**	h_w^* **mm**	b **mm**	t_w **mm**	t_f **mm**
500.0 x 320.0 x 100.01	33.47	500	320	5	16
550.0 x 100.0 x 29.44	4311.44	550	100	5	5
550.0 x 100.0 x 34.15	960.58	550	100	5	8
550.0 x 240.0 x 66.8	1057.55	550	240	5	12
550.0 x 280.0 x 83.13	871.98	550	280	5	14
600.0 x 100.0 x 31.4	3378.73	600	100	5	5
600.0 x 100.0 x 32.97	2488.46	600	100	5	6
600.0 x 100.0 x 36.11	1213.24	600	100	5	8
600.0 x 240.0 x 68.77	989.89	600	240	5	12
600.0 x 280.0 x 85.09	340.97	600	280	5	14
650.0 x 100.0 x 34.93	1466.33	650	100	5	6
650.0 x 100.0 x 38.07	1133.96	650	100	5	8
650.0 x 120.0 x 36.82	838.29	650	120	5	6
650.0 x 120.0 x 40.58	1016.51	650	120	5	8
650.0 x 280.0 x 87.06	264.45	650	280	5	14
650.0 x 320.0 x 105.9	26.59	650	320	5	16
700.0 x 120.0 x 38.78	452.08	700	120	5	6
700.0 x 320.0 x 107.86	0.91	700	320	5	16
750.0 x 100.0 x 42	613.43	750	100	5	8
750.0 x 100.0 x 48.28	135.86	750	100	5	12
750.0 x 100.0 x 51.42	9.06	750	100	5	14
750.0 x 120.0 x 44.51	742.86	750	120	5	8
750.0 x 120.0 x 48.28	135.86	750	120	5	10
750.0 x 140.0 x 51.42	9.06	750	140	5	10
750.0 x 280.0 x 90.98	387.31	750	280	5	14
800.0 x 100.0 x 50.24	64.92	800	100	5	12
800.0 x 120.0 x 46.47	284.45	800	120	5	8
800.0 x 120.0 x 50.24	64.92	800	120	5	10
800.0 x 140.0 x 53.38	6.39	800	140	5	10

Table E.9: Most optimal set of the final B3 Optimal Sets (obtained with a minimum rating of 90)

Section Name $h_w^* \times b \times m$	**Final ranking**	h_w^* **mm**	b **mm**	t_w **mm**	t_f **mm**	t_w/t_f	A_w/A	h_w^*/t_w
150.0 x 100.0 x 13.74	10293.60	150	100	5	5	1.00	0.43	30
150.0 x 100.0 x 16.48	428.42	150	100	6	6	1.00	0.43	25
150.0 x 100.0 x 24.73	245.00	150	100	5	12	0.42	0.24	30
150.0 x 120.0 x 17.19	979.89	150	120	5	6	0.83	0.34	30
150.0 x 120.0 x 20.96	564.41	150	120	5	8	0.63	0.28	30
200.0 x 100.0 x 15.7	1674.62	200	100	5	5	1.00	0.50	40
200.0 x 100.0 x 17.27	233.01	200	100	5	6	0.83	0.45	40
200.0 x 120.0 x 19.15	462.83	200	120	5	6	0.83	0.41	40
200.0 x 120.0 x 22.92	411.82	200	120	5	8	0.63	0.34	40
200.0 x 140.0 x 25.43	615.23	200	140	5	8	0.63	0.31	40
200.0 x 160.0 x 27.95	773.39	200	160	5	8	0.63	0.28	40
250.0 x 100.0 x 17.66	547.14	250	100	5	5	1.00	0.56	50
250.0 x 100.0 x 19.23	213.98	250	100	5	6	0.83	0.51	50
250.0 x 120.0 x 21.12	20.02	250	120	5	6	0.83	0.46	50
250.0 x 140.0 x 31.79	51.52	250	140	5	10	0.50	0.31	50
250.0 x 160.0 x 29.91	246.14	250	160	5	8	0.63	0.33	50
250.0 x 160.0 x 34.93	581.76	250	160	5	10	0.50	0.28	50
250.0 x 180.0 x 38.07	647.83	250	180	5	10	0.50	0.26	50
250.0 x 200.0 x 41.21	650.52	250	200	5	10	0.50	0.24	50
250.0 x 200.0 x 47.49	516.99	250	200	5	12	0.42	0.21	50
250.0 x 220.0 x 51.26	192.97	250	220	5	12	0.42	0.19	50
300.0 x 100.0 x 19.62	433.28	300	100	5	5	1.00	0.60	60
300.0 x 160.0 x 31.87	190.22	300	160	5	8	0.63	0.37	60
300.0 x 200.0 x 43.17	201.93	300	200	5	10	0.50	0.27	60
300.0 x 200.0 x 49.45	168.60	300	200	5	12	0.42	0.24	60
300.0 x 220.0 x 53.22	385.90	300	220	5	12	0.42	0.22	60
300.0 x 240.0 x 56.99	356.61	300	240	5	12	0.42	0.21	60
300.0 x 240.0 x 64.53	252.58	300	240	5	14	0.36	0.18	60
300.0 x 260.0 x 68.92	129.41	300	260	5	14	0.36	0.17	60
350.0 x 100.0 x 21.59	575.78	350	100	5	5	1.00	0.64	70

Note : The sections not highlighted can be used to replace the South African hot-rolled I-sections and the blue highlighted sections can be used to replace the South African plate girders.

Table E.9 (continued)

Section Name $h_w^* \times b \times m$	**Final ranking**	h_w^* **mm**	b **mm**	t_w **mm**	t_f **mm**	t_w/t_f	A_w/A	h_w^*/t_w
350.0 x 100.0 x 23.16	214.26	350	100	5	6	0.83	0.59	70
350.0 x 120.0 x 25.04	8.33	350	120	5	6	0.83	0.55	70
350.0 x 220.0 x 55.19	125.21	350	220	5	12	0.42	0.25	70
350.0 x 240.0 x 58.95	148.44	350	240	5	12	0.42	0.23	70
350.0 x 260.0 x 70.89	207.42	350	260	5	14	0.36	0.19	70
350.0 x 280.0 x 75.28	257.49	350	280	5	14	0.36	0.18	70
400.0 x 100.0 x 23.55	292.21	400	100	5	5	1.00	0.67	80
400.0 x 100.0 x 25.12	205.96	400	100	5	6	0.83	0.63	80
400.0 x 240.0 x 60.92	138.70	400	240	5	12	0.42	0.26	80
400.0 x 320.0 x 96.08	123.59	400	320	5	16	0.31	0.16	80
450.0 x 100.0 x 25.51	228.76	450	100	5	5	1.00	0.69	90
450.0 x 160.0 x 37.76	70.82	450	160	5	8	0.63	0.47	90
500.0 x 100.0 x 27.48	375.37	500	100	5	5	1.00	0.71	100
500.0 x 320.0 x 100.01	21.86	500	320	5	16	0.31	0.20	100
550.0 x 100.0 x 29.44	323.32	550	100	5	5	1.00	0.73	110
550.0 x 240.0 x 66.8	126.33	550	240	5	12	0.42	0.32	110
550.0 x 280.0 x 83.13	248.81	550	280	5	14	0.36	0.26	110
600.0 x 100.0 x 31.4	241.04	600	100	5	5	1.00	0.75	120
600.0 x 100.0 x 36.11	161.32	600	100	5	8	0.63	0.65	120
600.0 x 320.0 x 103.93	10.02	600	320	5	16	0.31	0.23	120
650.0 x 100.0 x 33.36	175.83	650	100	5	5	1.00	0.76	130
650.0 x 100.0 x 38.07	128.07	650	100	5	8	0.63	0.67	130
650.0 x 100.0 x 41.21	19.78	650	100	5	10	0.50	0.62	130
650.0 x 280.0 x 87.06	84.46	650	280	5	14	0.36	0.29	130
700.0 x 100.0 x 40.03	72.13	700	100	5	8	0.63	0.69	140
700.0 x 120.0 x 42.55	36.22	700	120	5	8	0.63	0.65	140
700.0 x 180.0 x 55.73	1.82	700	180	5	10	0.50	0.49	140
700.0 x 200.0 x 58.88	0.87	700	200	5	10	0.50	0.47	140
750.0 x 160.0 x 49.53	16.04	750	160	5	8	0.63	0.59	150
750.0 x 200.0 x 60.84	0.02	750	200	5	10	0.50	0.48	150
750.0 x 320.0 x 109.82	5.66	750	320	5	16	0.31	0.27	150

Note : The sections not highlighted can be used to replace the South African hot-rolled I-sections and the blue highlighted sections can be used to replace the South African plate girders.

Table E.9 (continued)

Section Name $h_w^* \times b \times m$	**Final ranking**	h_w^* **mm**	b **mm**	t_w **mm**	t_f **mm**	t_w/t_f	A_w/A	h_w^*/t_w
800.0 x 120.0 x 46.47	49.47	800	120	5	8	0.63	0.68	160
850.0 x 100.0 x 52.2	6.19	850	100	5	12	0.42	0.64	170
850.0 x 120.0 x 52.2	6.19	850	120	5	10	0.50	0.64	170
850.0 x 320.0 x 113.75	0.43	850	320	5	16	0.31	0.29	170

Note : The sections not highlighted can be used to replace the South African hot-rolled I-sections and the blue highlighted sections can be used to replace the South African plate girders.

Table E.10: Most practical set of the final B3 Optimal Sets (obtained with a minimum rating of 80)

Section Name $h_w^* \times b \times m$	**Final ranking**	h_w^* **mm**	b **mm**	t_w **mm**	t_f **mm**	t_w/t_f	A_w/A	h_w^*/t_w
150.0 x 100.0 x 18.45	98.74	150	100	5	8	0.63	0.32	30
150.0 x 100.0 x 21.59	305.30	150	100	5	10	0.50	0.27	30
150.0 x 100.0 x 24.73	245.00	150	100	5	12	0.42	0.24	30
150.0 x 120.0 x 17.19	1408.31	150	120	5	6	0.83	0.34	30
200.0 x 100.0 x 15.7	11968.21	200	100	5	5	1.00	0.50	40
200.0 x 120.0 x 19.15	364.10	200	120	5	6	0.83	0.41	40
200.0 x 120.0 x 22.92	342.87	200	120	5	8	0.63	0.34	40
200.0 x 160.0 x 27.95	512.60	200	160	5	8	0.63	0.28	40
200.0 x 160.0 x 32.97	374.05	200	160	5	10	0.50	0.24	40
250.0 x 100.0 x 17.66	780.15	250	100	5	5	1.00	0.56	50
250.0 x 120.0 x 21.12	348.08	250	120	5	6	0.83	0.46	50
250.0 x 120.0 x 24.88	44.17	250	120	5	8	0.63	0.39	50
250.0 x 140.0 x 27.4	667.05	250	140	5	8	0.63	0.36	50
250.0 x 160.0 x 29.91	246.14	250	160	5	8	0.63	0.33	50
250.0 x 160.0 x 34.93	207.84	250	160	5	10	0.50	0.28	50
250.0 x 180.0 x 38.07	647.83	250	180	5	10	0.50	0.26	50
250.0 x 200.0 x 41.21	650.52	250	200	5	10	0.50	0.24	50
250.0 x 200.0 x 47.49	332.03	250	200	5	12	0.42	0.21	50
300.0 x 100.0 x 19.62	647.26	300	100	5	5	1.00	0.60	60
300.0 x 100.0 x 21.2	217.73	300	100	5	6	0.83	0.56	60

Note : The sections not highlighted can be used to replace the South African hot-rolled I-sections and the blue highlighted sections can be used to replace the South African plate girders.

Table E.10 (continued)

Section Name $h_w^* \times b \times m$	**Final ranking**	h_w^* **mm**	b **mm**	t_w **mm**	t_f **mm**	t_w/t_f	A_w/A	h_w^*/t_w
300.0 x 120.0 x 23.08	219.31	300	120	5	6	0.83	0.51	60
300.0 x 120.0 x 26.85	164.80	300	120	5	8	0.63	0.44	60
300.0 x 160.0 x 31.87	241.60	300	160	5	8	0.63	0.37	60
300.0 x 220.0 x 53.22	418.66	300	220	5	12	0.42	0.22	60
300.0 x 240.0 x 56.99	480.05	300	240	5	12	0.42	0.21	60
350.0 x 120.0 x 25.04	10.07	350	120	5	6	0.83	0.55	70
350.0 x 200.0 x 45.14	386.90	350	200	5	10	0.50	0.30	70
350.0 x 280.0 x 75.28	169.59	350	280	5	14	0.36	0.18	70
400.0 x 100.0 x 23.55	643.48	400	100	5	5	1.00	0.67	80
400.0 x 100.0 x 25.12	205.96	400	100	5	6	0.83	0.63	80
400.0 x 120.0 x 27	16.65	400	120	5	6	0.83	0.58	80
400.0 x 240.0 x 60.92	287.14	400	240	5	12	0.42	0.26	80
400.0 x 260.0 x 72.85	238.54	400	260	5	14	0.36	0.22	80
450.0 x 100.0 x 25.51	228.76	450	100	5	5	1.00	0.69	90
450.0 x 160.0 x 37.76	70.82	450	160	5	8	0.63	0.47	90
500.0 x 100.0 x 27.48	358.72	500	100	5	5	1.00	0.71	100
500.0 x 300.0 x 94.98	104.86	500	300	5	16	0.31	0.21	100
550.0 x 100.0 x 29.44	323.32	550	100	5	5	1.00	0.73	110
550.0 x 140.0 x 39.17	17.83	550	140	5	8	0.63	0.55	110
550.0 x 160.0 x 41.68	1.31	550	160	5	8	0.63	0.52	110
550.0 x 200.0 x 52.99	322.11	550	200	5	10	0.50	0.41	110
550.0 x 240.0 x 66.8	378.91	550	240	5	12	0.42	0.32	110
550.0 x 320.0 x 101.97	46.49	550	320	5	16	0.31	0.21	110
600.0 x 100.0 x 31.4	241.04	600	100	5	5	1.00	0.75	120
600.0 x 100.0 x 32.97	139.26	600	100	5	6	0.83	0.71	120
600.0 x 120.0 x 34.85	128.95	600	120	5	6	0.83	0.68	120
600.0 x 180.0 x 51.81	6.70	600	180	5	10	0.50	0.45	120
650.0 x 120.0 x 36.82	136.70	650	120	5	6	0.83	0.69	130
650.0 x 160.0 x 45.61	19.04	650	160	5	8	0.63	0.56	130
650.0 x 160.0 x 50.63	5.40	650	160	5	10	0.50	0.50	130
650.0 x 180.0 x 53.77	7.58	650	180	5	10	0.50	0.47	130

Note : The sections not highlighted can be used to replace the South African hot-rolled I-sections and the blue highlighted sections can be used to replace the South African plate girders.

Table E.10 (continued)

Section Name $h_w^* \times b \times m$	Final ranking	h_w^* mm	b mm	t_w mm	t_f mm	t_w/t_f	A_w/A	h_w^*/t_w
650.0 x 200.0 x 56.91	1.76	650	200	5	10	0.50	0.45	130
650.0 x 200.0 x 63.19	0.02	650	200	5	12	0.42	0.40	130
650.0 x 240.0 x 70.73	186.19	650	240	5	12	0.42	0.36	130
650.0 x 280.0 x 87.06	333.27	650	280	5	14	0.36	0.29	130
700.0 x 100.0 x 40.03	114.61	700	100	5	8	0.63	0.69	140
700.0 x 100.0 x 43.17	66.46	700	100	5	10	0.50	0.64	140
700.0 x 160.0 x 47.57	25.22	700	160	5	8	0.63	0.58	140
700.0 x 180.0 x 55.73	4.51	700	180	5	10	0.50	0.49	140
700.0 x 200.0 x 58.88	0.87	700	200	5	10	0.50	0.47	140
800.0 x 320.0 x 111.78	10.19	800	320	5	16	0.31	0.28	160

Note : The sections not highlighted can be used to replace the South African hot-rolled I-sections and the blue highlighted sections can be used to replace the South African plate girders.

E.1.4 Based on Initial Set 4

Table E.11: Sections present in all of the final C4 Optimal Sets

Section Name $h_w^* \times b \times m$	h_w^* mm	b mm	t_w mm	t_f mm
250.0 x 120.0 x 24.88	250	120	5	8
250.0 x 140.0 x 27.4	250	140	5	8
250.0 x 160.0 x 29.91	250	160	5	8
300.0 x 100.0 x 21.2	300	100	5	6
300.0 x 180.0 x 40.03	300	180	5	10
350.0 x 100.0 x 23.16	350	100	5	6
350.0 x 140.0 x 31.32	350	140	5	8
350.0 x 140.0 x 34.07	350	140	6	8
400.0 x 100.0 x 25.12	400	100	5	6
400.0 x 140.0 x 33.28	400	140	5	8
400.0 x 180.0 x 47.1	400	180	6	10
450.0 x 100.0 x 30.22	450	100	5	8
450.0 x 140.0 x 35.25	450	140	5	8
450.0 x 160.0 x 37.76	450	160	5	8

Table E.11 (continued)

Section Name $h_w^* \times b \times m$	h_w^* mm	b mm	t_w mm	t_f mm
450.0 x 160.0 x 46.31	450	160	6	10
450.0 x 180.0 x 49.45	450	180	6	10
450.0 x 200.0 x 49.06	450	200	5	10
450.0 x 200.0 x 52.59	450	200	6	10
450.0 x 220.0 x 62.64	450	220	6	12
500.0 x 160.0 x 39.72	500	160	5	8
500.0 x 220.0 x 65	500	220	6	12
550.0 x 120.0 x 32.89	550	120	5	6
550.0 x 220.0 x 67.35	550	220	6	12
600.0 x 120.0 x 34.85	600	120	5	6
600.0 x 160.0 x 43.65	600	160	5	8
600.0 x 200.0 x 54.95	600	200	5	10
600.0 x 220.0 x 69.71	600	220	6	12
650.0 x 160.0 x 45.61	650	160	5	8
650.0 x 220.0 x 72.06	650	220	6	12
750.0 x 160.0 x 49.53	750	160	5	8
750.0 x 200.0 x 60.84	750	200	5	10
800.0 x 160.0 x 57.78	800	160	6	8
800.0 x 220.0 x 72.85	800	220	5	12
800.0 x 240.0 x 82.9	800	240	6	12
800.0 x 260.0 x 94.83	800	260	6	14
850.0 x 200.0 x 64.76	850	200	5	10
900.0 x 200.0 x 114.61	900	200	10	14
900.0 x 240.0 x 87.61	900	240	6	12
900.0 x 240.0 x 95.14	900	240	6	14
900.0 x 260.0 x 135.96	900	260	10	16
950.0 x 200.0 x 76.14	950	200	6	10
950.0 x 220.0 x 78.74	950	220	5	12
950.0 x 220.0 x 93.1	950	220	6	14
950.0 x 240.0 x 89.96	950	240	6	12
950.0 x 240.0 x 97.5	950	240	6	14
950.0 x 260.0 x 101.89	950	260	6	14

Table E.11 (continued)

Section Name $h_w^* \times b \times m$	h_w^* **mm**	b **mm**	t_w **mm**	t_f **mm**
950.0 x 260.0 x 110.06	950	260	6	16
950.0 x 260.0 x 94.44	950	260	5	14
950.0 x 280.0 x 106.29	950	280	6	14
950.0 x 280.0 x 107.62	950	280	5	16
950.0 x 280.0 x 115.08	950	280	6	16
950.0 x 280.0 x 121.2	950	280	8	14
950.0 x 280.0 x 130	950	280	8	16
950.0 x 300.0 x 112.65	950	300	5	16
950.0 x 300.0 x 120.11	950	300	6	16
950.0 x 300.0 x 135.02	950	300	8	16
950.0 x 320.0 x 125.13	950	320	6	16
950.0 x 320.0 x 135.18	950	320	6	18
950.0 x 320.0 x 140.04	950	320	8	16
950.0 x 340.0 x 140.83	950	340	6	18
950.0 x 340.0 x 151.5	950	340	6	20
1150.0 x 260.0 x 110.45	1150	260	5	16
1150.0 x 280.0 x 151.82	1150	280	10	14
1200.0 x 280.0 x 155.74	1200	280	10	14
1250.0 x 240.0 x 131.25	1250	240	8	14
1250.0 x 280.0 x 159.67	1250	280	10	14
1300.0 x 240.0 x 106.45	1300	240	6	12
1300.0 x 260.0 x 118.38	1300	260	6	14
1350.0 x 280.0 x 146.32	1350	280	8	14
1350.0 x 300.0 x 160.14	1350	300	8	16
1500.0 x 320.0 x 198.13	1500	320	10	16
1550.0 x 300.0 x 172.7	1550	300	8	16
1700.0 x 380.0 x 240.84	1700	380	10	18

Table E.12: Most optimal set of the final C4 Optimal Sets

Section Name $h_w^* \times b \times m$	**Final ranking**	h_w^* **mm**	b **mm**	t_w **mm**	t_f **mm**	t_w/t_f	A_w/A	h_w^*/t_w
250.0 x 120.0 x 23.08	140.41	250	120	6	6	1.00	0.51	42
250.0 x 120.0 x 24.88	222.27	250	120	5	8	0.63	0.39	50
250.0 x 120.0 x 28.65	15.13	250	120	5	10	0.50	0.34	50
250.0 x 140.0 x 27.4	331.35	250	140	5	8	0.63	0.36	50
250.0 x 160.0 x 29.91	185.63	250	160	5	8	0.63	0.33	50
300.0 x 100.0 x 19.62	1280.78	300	100	5	5	1.00	0.60	60
300.0 x 100.0 x 21.2	471.12	300	100	5	6	0.83	0.56	60
300.0 x 120.0 x 25.43	1.37	300	120	6	6	1.00	0.56	50
300.0 x 140.0 x 31.71	10.81	300	140	6	8	0.75	0.45	50
300.0 x 140.0 x 36.11	14.72	300	140	6	10	0.60	0.39	50
300.0 x 180.0 x 40.03	114.46	300	180	5	10	0.50	0.29	60
350.0 x 100.0 x 23.16	437.89	350	100	5	6	0.83	0.59	70
350.0 x 100.0 x 29.05	6.67	350	100	6	8	0.75	0.57	58
350.0 x 140.0 x 31.32	198.77	350	140	5	8	0.63	0.44	70
350.0 x 140.0 x 34.07	9.36	350	140	6	8	0.75	0.48	58
400.0 x 100.0 x 25.12	426.62	400	100	5	6	0.83	0.63	80
400.0 x 100.0 x 28.26	481.51	400	100	5	8	0.63	0.56	80
400.0 x 120.0 x 30.77	248.83	400	120	5	8	0.63	0.51	80
400.0 x 120.0 x 37.68	37.50	400	120	6	10	0.60	0.50	67
400.0 x 140.0 x 33.28	222.54	400	140	5	8	0.63	0.47	80
400.0 x 160.0 x 40.82	44.31	400	160	5	10	0.50	0.38	80
400.0 x 160.0 x 43.96	78.83	400	160	6	10	0.60	0.43	67
400.0 x 180.0 x 47.1	156.12	400	180	6	10	0.60	0.40	67
450.0 x 100.0 x 30.22	444.46	450	100	5	8	0.63	0.58	90
450.0 x 120.0 x 32.5	1.81	450	120	6	6	1.00	0.65	75
450.0 x 140.0 x 35.25	503.78	450	140	5	8	0.63	0.50	90
450.0 x 160.0 x 37.76	442.36	450	160	5	8	0.63	0.47	90
450.0 x 160.0 x 46.31	44.17	450	160	6	10	0.60	0.46	75
450.0 x 180.0 x 49.45	25.64	450	180	6	10	0.60	0.43	75
450.0 x 180.0 x 55.11	41.95	450	180	6	12	0.50	0.38	75
450.0 x 200.0 x 49.06	424.05	450	200	5	10	0.50	0.36	90

Note : The sections not highlighted can be used to replace the South African plate girders and the blue highlighted sections can be used to replace the South African hot-rolled I-sections.

Table E.12 (continued)

Section Name $h_w^* \times b \times m$	Final ranking	h_w^* mm	b mm	t_w mm	t_f mm	t_w/t_f	A_w/A	h_w^*/t_w
450.0 x 200.0 x 52.59	25.84	450	200	6	10	0.60	0.40	75
450.0 x 220.0 x 62.64	55.61	450	220	6	12	0.50	0.34	75
500.0 x 160.0 x 39.72	196.55	500	160	5	8	0.63	0.49	100
500.0 x 220.0 x 65	38.99	500	220	6	12	0.50	0.36	83
550.0 x 120.0 x 32.89	203.37	550	120	5	6	0.83	0.66	110
550.0 x 140.0 x 39.17	406.21	550	140	5	8	0.63	0.55	110
550.0 x 180.0 x 62.8	5.85	550	180	8	10	0.80	0.55	69
550.0 x 220.0 x 67.35	57.71	550	220	6	12	0.50	0.38	92
600.0 x 120.0 x 34.85	214.82	600	120	5	6	0.83	0.68	120
600.0 x 140.0 x 41.13	250.84	600	140	5	8	0.63	0.57	120
600.0 x 160.0 x 43.65	495.81	600	160	5	8	0.63	0.54	120
600.0 x 160.0 x 62.8	11.53	600	160	8	10	0.80	0.60	75
600.0 x 180.0 x 51.81	229.67	600	180	5	10	0.50	0.45	120
600.0 x 200.0 x 54.95	566.56	600	200	5	10	0.50	0.43	120
600.0 x 220.0 x 69.71	41.14	600	220	6	12	0.50	0.41	100
650.0 x 160.0 x 45.61	421.74	650	160	5	8	0.63	0.56	130
650.0 x 160.0 x 65.94	25.50	650	160	8	10	0.80	0.62	81
650.0 x 200.0 x 56.91	234.03	650	200	5	10	0.50	0.45	130
650.0 x 220.0 x 72.06	61.78	650	220	6	12	0.50	0.42	108
700.0 x 180.0 x 72.22	2.90	700	180	8	10	0.80	0.61	88
700.0 x 220.0 x 74.42	20.34	700	220	6	12	0.50	0.44	117
700.0 x 240.0 x 78.19	79.53	700	240	6	12	0.50	0.42	117
750.0 x 160.0 x 49.53	456.45	750	160	5	8	0.63	0.59	150
750.0 x 200.0 x 60.84	353.27	750	200	5	10	0.50	0.48	150
750.0 x 240.0 x 80.54	30.68	750	240	6	12	0.50	0.44	125
800.0 x 160.0 x 57.78	43.70	800	160	6	8	0.75	0.65	133
800.0 x 220.0 x 72.85	239.64	800	220	5	12	0.42	0.43	160
800.0 x 240.0 x 82.9	66.55	800	240	6	12	0.50	0.45	133
800.0 x 260.0 x 94.83	23.22	800	260	6	14	0.43	0.40	133
850.0 x 120.0 x 55.97	321.63	850	120	5	12	0.42	0.60	170
850.0 x 160.0 x 58.48	150.61	850	160	5	10	0.50	0.57	170

Note : The sections not highlighted can be used to replace the South African plate girders and the blue highlighted sections can be used to replace the South African hot-rolled I-sections.

Table E.12 (continued)

Section Name $h_w^* \times b \times m$	**Final ranking**	h_w^* **mm**	b **mm**	t_w **mm**	t_f **mm**	t_w/t_f	A_w/A	h_w^*/t_w
850.0 x 200.0 x 64.76	534.80	850	200	5	10	0.50	0.52	170
900.0 x 100.0 x 86.35	1.89	900	100	10	10	1.00	0.82	90
900.0 x 160.0 x 67.51	27.13	900	160	6	10	0.60	0.63	150
900.0 x 200.0 x 114.61	0.74	900	200	10	14	0.71	0.62	90
900.0 x 220.0 x 119.01	3.46	900	220	10	14	0.71	0.59	90
900.0 x 220.0 x 125.91	6.15	900	220	10	16	0.63	0.56	90
900.0 x 240.0 x 87.61	68.35	900	240	6	12	0.50	0.48	150
900.0 x 240.0 x 95.14	5.28	900	240	6	14	0.43	0.45	150
900.0 x 260.0 x 135.96	3.65	900	260	10	16	0.63	0.52	90
900.0 x 280.0 x 149.78	2.69	900	280	10	18	0.56	0.47	90
950.0 x 200.0 x 68.69	342.40	950	200	5	10	0.50	0.54	190
950.0 x 200.0 x 76.14	92.95	950	200	6	10	0.60	0.59	158
950.0 x 220.0 x 78.74	289.59	950	220	5	12	0.42	0.47	190
950.0 x 220.0 x 85.64	248.94	950	220	5	14	0.36	0.44	190
950.0 x 220.0 x 93.1	68.22	950	220	6	14	0.43	0.48	158
950.0 x 240.0 x 89.96	54.98	950	240	6	12	0.50	0.50	158
950.0 x 240.0 x 97.5	28.59	950	240	6	14	0.43	0.46	158
950.0 x 260.0 x 101.89	42.39	950	260	6	14	0.43	0.44	158
950.0 x 260.0 x 102.6	46.27	950	260	5	16	0.31	0.36	190
950.0 x 260.0 x 110.06	14.01	950	260	6	16	0.38	0.41	158
950.0 x 260.0 x 116.81	2.86	950	260	8	14	0.57	0.51	119
950.0 x 260.0 x 94.44	236.83	950	260	5	14	0.36	0.39	190
950.0 x 280.0 x 106.29	37.86	950	280	6	14	0.43	0.42	158
950.0 x 280.0 x 107.62	33.88	950	280	5	16	0.31	0.35	190
950.0 x 280.0 x 115.08	36.98	950	280	6	16	0.38	0.39	158
950.0 x 280.0 x 121.2	1.80	950	280	8	14	0.57	0.49	119
950.0 x 280.0 x 130	13.05	950	280	8	16	0.50	0.46	119
950.0 x 280.0 x 132.66	3.62	950	280	6	20	0.30	0.34	158
950.0 x 300.0 x 112.65	23.84	950	300	5	16	0.31	0.33	190
950.0 x 300.0 x 120.11	18.42	950	300	6	16	0.38	0.37	158
950.0 x 300.0 x 135.02	14.14	950	300	8	16	0.50	0.44	119

Note : The sections not highlighted can be used to replace the South African plate girders and the blue highlighted sections can be used to replace the South African hot-rolled I-sections.

Table E.12 (continued)

Section Name $h_w^* \times b \times m$	**Final ranking**	h_w^* **mm**	b **mm**	t_w **mm**	t_f **mm**	t_w/t_f	A_w/A	h_w^*/t_w
950.0 x 320.0 x 125.13	19.56	950	320	6	16	0.38	0.36	158
950.0 x 320.0 x 127.72	2.99	950	320	5	18	0.28	0.29	190
950.0 x 320.0 x 135.18	7.61	950	320	6	18	0.33	0.33	158
950.0 x 320.0 x 140.04	13.48	950	320	8	16	0.50	0.43	119
950.0 x 320.0 x 150.09	4.92	950	320	8	18	0.44	0.40	119
950.0 x 320.0 x 160.14	5.26	950	320	8	20	0.40	0.37	119
950.0 x 340.0 x 140.83	6.81	950	340	6	18	0.33	0.32	158
950.0 x 340.0 x 151.5	4.68	950	340	6	20	0.30	0.30	158
950.0 x 380.0 x 167.05	1.83	950	380	8	18	0.44	0.36	119
1050.0 x 200.0 x 87.14	21.49	1050	200	6	12	0.50	0.57	175
1050.0 x 220.0 x 147.27	5.77	1050	220	12	14	0.86	0.67	88
1100.0 x 160.0 x 81.95	11.28	1100	160	6	12	0.50	0.63	183
1100.0 x 240.0 x 95.93	55.98	1100	240	5	14	0.36	0.45	220
1100.0 x 320.0 x 166.73	4.00	1100	320	10	16	0.63	0.52	110
1150.0 x 180.0 x 73.4	117.28	1150	180	5	10	0.50	0.61	230
1150.0 x 260.0 x 110.45	32.22	1150	260	5	16	0.31	0.41	230
1150.0 x 280.0 x 151.82	2.35	1150	280	10	14	0.71	0.59	115
1200.0 x 240.0 x 101.74	56.39	1200	240	6	12	0.50	0.56	200
1200.0 x 280.0 x 136.9	9.94	1200	280	8	14	0.57	0.55	150
1200.0 x 280.0 x 155.74	7.78	1200	280	10	14	0.71	0.60	120
1200.0 x 300.0 x 169.56	3.37	1200	300	10	16	0.63	0.56	120
1250.0 x 240.0 x 131.25	8.83	1250	240	8	14	0.57	0.60	156
1250.0 x 280.0 x 140.04	10.17	1250	280	8	14	0.57	0.56	156
1250.0 x 280.0 x 159.67	4.18	1250	280	10	14	0.71	0.61	125
1300.0 x 240.0 x 106.45	45.63	1300	240	6	12	0.50	0.58	217
1300.0 x 240.0 x 113.98	24.90	1300	240	6	14	0.43	0.54	217
1300.0 x 260.0 x 118.38	31.59	1300	260	6	14	0.43	0.52	217
1300.0 x 320.0 x 182.43	13.95	1300	320	10	16	0.63	0.56	130
1350.0 x 280.0 x 133.92	4.54	1350	280	6	16	0.38	0.47	225
1350.0 x 280.0 x 146.32	18.85	1350	280	8	14	0.57	0.58	169
1350.0 x 300.0 x 160.14	19.19	1350	300	8	16	0.50	0.53	169

Note : The sections not highlighted can be used to replace the South African plate girders and the blue highlighted sections can be used to replace the South African hot-rolled I-sections.

Table E.12 (continued)

Section Name $h_w^* \times b \times m$	**Final ranking**	h_w^* **mm**	b **mm**	t_w **mm**	t_f **mm**	t_w/t_f	A_w/A	h_w^*/t_w
1350.0 x 320.0 x 165.16	13.48	1350	320	8	16	0.50	0.51	169
1350.0 x 360.0 x 165.32	4.51	1350	360	6	18	0.33	0.38	225
1400.0 x 260.0 x 123.09	24.32	1400	260	6	14	0.43	0.54	233
1400.0 x 260.0 x 131.25	25.33	1400	260	6	16	0.38	0.50	233
1400.0 x 320.0 x 146.32	18.88	1400	320	6	16	0.38	0.45	233
1450.0 x 280.0 x 152.6	12.27	1450	280	8	14	0.57	0.60	181
1500.0 x 320.0 x 198.13	14.55	1500	320	10	16	0.63	0.59	150
1550.0 x 300.0 x 172.7	11.08	1550	300	8	16	0.50	0.56	194
1600.0 x 320.0 x 205.98	7.53	1600	320	10	16	0.63	0.61	160
1700.0 x 320.0 x 187.14	15.87	1700	320	8	16	0.50	0.57	213
1700.0 x 380.0 x 240.84	4.62	1700	380	10	18	0.56	0.55	170
1750.0 x 320.0 x 217.76	9.07	1750	320	10	16	0.63	0.63	175
1850.0 x 280.0 x 204.1	3.54	1850	280	8	20	0.40	0.57	231
1850.0 x 320.0 x 225.61	4.12	1850	320	10	16	0.63	0.64	185
1850.0 x 340.0 x 222.94	1.65	1850	340	8	20	0.40	0.52	231
1950.0 x 320.0 x 263.6	0.11	1950	320	10	22	0.45	0.58	195

Note : The sections not highlighted can be used to replace the South African plate girders and the blue highlighted sections can be used to replace the South African hot-rolled I-sections.

Appendix F

South African I-sections for Design Space A, B and C

Table F.1: The South African hot-rolled I-sections and plate girders which work for Design Space A, B and C

Section Name $h_w^* \times b \times m$	**Weighting A Final ranking**	**Weighting B Final ranking**	**Weighting C Final ranking**	h_w^* **mm**	b **mm**	t_w **mm**	t_f **mm**
IPE-AA 100	143.05	3117.90	5.15	88.6	55	3.6	4.5
IPE 100	194.45	1005.49		88.6	55	4.1	5.7
IPE-AA 120	312.43	753.89	18.29	107.4	64	3.8	4.8
IPE 120	29.40	21.63		107.4	64	4.4	6.3
IPE-AA 140	1318.85	2089.78	50.29	126.2	73	3.8	5.2
IPE-AA 160	2775.72	2285.54	114.40	145.2	82	4	5.6
IPE 160	175.20	88.03		145.2	82	5	7.4
IPE-AA 180	4471.78	2145.68	228.07	164	91	4.3	6.2
IPE 180	54.00	8.00		164	91	5.3	8
IPE-AA 200	6347.41	1878.14	340.93	183	100	4.5	6.7
IPE 200	6323.73	1455.68	428.56	183	100	5.6	8.5
152x152x23			2.84	138.8	152.4	6.1	6.8
203x133x25	4827.38	1092.63	110.70	187.6	133.4	5.7	7.8
203x133x30	4726.08	877.98	142.57	187.6	133.8	6.4	9.6
254x146x31	1380.46	197.15	262.99	234.3	146.1	6	8.6
254x146x37	6057.00	978.63	387.67	234.2	146.4	6.3	10.9
254x146x43	960.59	158.82		234.2	147.3	7.2	12.7
254x254x73	2.80	3.42		225.8	254	8.6	14.2
305x102x25	13814.02	1295.10	950.88	290.8	101.6	5.8	7

Note : If the section has a blank space as a final ranking, the section was not sufficient for the corresponding design space.

Table F.1 (continued)

Section Name $h_w^* \times b \times m$	**Weighting A Final ranking**	**Weighting B Final ranking**	**Weighting C Final ranking**	h_w^* **mm**	b **mm**	t_w **mm**	t_f **mm**
305x102x28	4337.15	350.25	504.28	291.3	101.9	6	8.8
305x102x33	3858.56	291.88	429.56	291.1	102.4	6.6	10.8
305x165x40	4026.95	573.66	277.37	283.4	165.1	6.1	10.2
305x165x46	2390.30	326.69	47.02	283.5	165.7	6.7	11.8
305x165x54	3177.76	420.95	51.09	283.5	166.8	7.7	13.7
356x171x45	1312.46	148.81	416.32	332.6	171	6.9	9.7
356x171x51	2144.98	309.21	321.76	332.6	171.5	7.3	11.5
356x171x57	1220.05	143.74	39.57	332.6	172.1	8	13
356x171x67	4891.79	615.31		332.6	173.2	9.1	15.7
406x140x39	13960.37	1007.89	1812.97	380.1	141.8	6.3	8.6
406x140x46	5005.02	329.71	651.39	379.9	142.4	6.9	11.2
406x178x54	3374.96	213.63	719.87	380.8	177.6	7.6	10.9
406x178x60	4145.91	366.95	725.68	380.8	177.8	7.8	12.8
406x178x67	5814.88	603.45		380.8	178.8	8.8	14.3
406x178x74	3254.63	387.18	140.65	380.8	179.7	9.7	16
457x191x67	8219.88	764.08	1136.70	428.2	189.9	8.5	12.7
457x191x74	1098.90	70.03	497.81	428.2	190.5	9.1	14.5
457x191x82	1861.62	266.18		428.2	191.3	9.9	16
457x191x89	3114.25	361.73	42.22	428.2	192	10.6	17.7
533x210x101	32.48	17.30		501.9	210.1	10.9	17.4
533x210x82	151.92	6.84	7.36	501.9	208.7	9.6	13.2
704 x 200 (8W, 12F)	2504.00	225.09	1991.13	680	200	8	12
704 x 250 (8W, 12F)	4162.05	429.64	504.13	680	250	8	12
712 x 200 (8W, 16F)			350.91	680	200	8	16
712 x 250 (8W, 16F)	1203.72	160.67	61.91	680	250	8	16
712 x 300 (8W, 16F)	120.61	31.56		680	300	8	16
720 x 300 (8W, 20F)		1.58		680	300	8	20
812 x 200 (8W, 16F)			118.60	780	200	8	16
812 x 250 (8W, 16F)			17.78	780	250	8	16
812 x 300 (8W, 16F)	19.18	7.87	29.31	780	300	8	16
820 x 250 (8W, 20F)			3.60	780	250	8	20

Note : If the section has a blank space as a final ranking, the section was not sufficient for the corresponding design space.

Table F.1 (continued)

Section Name $h_w^* \times b \times m$	**Weighting A Final ranking**	**Weighting B Final ranking**	**Weighting C Final ranking**	h_w^* mm	b mm	t_w mm	t_f mm
820 x 300 (8W, 20F)			3.70	780	300	8	20
830 x 250 (8W, 25F)			6.85	780	250	8	25
830 x 300 (8W, 25F)			2.68	780	300	8	25
904 x 200 (8W, 12F)			325.73	880	200	8	12
904 x 250 (8W, 12F)	2042.47	234.84	411.75	880	250	8	12
904 x 300 (8W, 12F)	1059.84	171.36	112.95	880	300	8	12
912 x 200 (8W, 16F)			55.34	880	200	8	16
912 x 250 (8W, 16F)			106.08	880	250	8	16
912 x 300 (8W, 16F)	4.57	3.78	76.43	880	300	8	16
920 x 300 (8W, 20F)			36.34	880	300	8	20
930 x 250 (8W, 25F)			4.25	880	250	8	25
930 x 300 (8W, 25F)			3.46	880	300	8	25
1004 x 250 (8W, 12F)			148.43	980	250	8	12
1012 x 250 (8W, 16F)			23.18	980	250	8	16
1012 x 300 (8W, 16F)			90.53	980	300	8	16
1020 x 400 (8W, 20F)			2.40	980	400	8	20
1204 x 250 (8W, 12F)			146.35	1180	250	8	12
1212 x 250 (8W, 16F)			109.27	1180	250	8	16
1212 x 300 (8W, 16F)			88.14	1180	300	8	16
1220 x 300 (8W, 20F)			45.49	1180	300	8	20
1220 x 400 (8W, 20F)			8.91	1180	400	8	20
1230 x 250 (8W, 25F)			7.73	1180	250	8	25
1230 x 300 (8W, 25F)			10.70	1180	300	8	25
1240 x 300 (8W, 30F)			2.28	1180	300	8	30
1402 x 250 (10W, 16F)			28.02	1370	250	10	16
1402 x 300 (10W, 16F)			32.92	1370	300	10	16
1410 x 300 (10W, 20F)			13.11	1370	300	10	20
1410 x 400 (10W, 20F)			7.20	1370	400	10	20
1420 x 250 (10W, 25F)			1.23	1370	250	10	25
1420 x 300 (10W, 25F)			7.32	1370	300	10	25

Note : If the section has a blank space as a final ranking, the section was not sufficient for the corresponding design space.

Table F.1 (continued)

Section Name $h_w^* \times b \times m$	**Weighting A Final ranking**	**Weighting B Final ranking**	**Weighting C Final ranking**	h_w^* **mm**	b **mm**	t_w **mm**	t_f **mm**
1420 x 400 (10W, 25F)			1.71	1370	400	10	25
1430 x 300 (10W, 30F)			4.39	1370	300	10	30
1602 x 300 (12W, 16F)			12.06	1570	300	12	16

Note : If the section has a blank space as a final ranking, the section was not sufficient for the corresponding design space.